University of Southern Queensland

Faculty of Engineering and Surveying

Pile Driving Analysis Via Dynamic Loading Test

A dissertation submitted by

Mr. Yu Wah, YU

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ABSTRACT

During the pile driving, there are many problems to be concerned such as pile damage, hammer performance to the pile, drivability of pile in various soil strata and determine pile bearing capacity. Unfortunately, being the pile driven underground and therefore 'out of sight' identifying those problems and therefore implementing remedial actions is considerably more difficult than in structure placed above ground level.

This research gives an overview of most common testing method used to determine the cause of pile damage and the pile bearing capacity during pile driving. By WAVE equation, it can be predicted the desired pile bearing capacity with applicable hammer configuration. However, this process is before pile driving and therefore the problems are still 'out of sight'. Thereafter, Case method is developed to monitor and identify the pile defects and pile bearing capacity during pile driving and therefore those problems will not be 'out of mind'.

Applying case histories to build up a statistically results, the paper also describes the relationship between pile damage and pile bearing capacity.

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Yu Yu Wah

Student number: 0050016432

CANDIDATES CERTIFICATION

I certify that the ideas, designs and experimental work, results, analysis and conclusions set out in this dissertation are entirely my own efforts, except where otherwise indicated and acknowledged.

I further certify that the work is original and has not been previously submitted for assessment in any other course or institution, except where specifically stated.

Yu Yu Wah

Student Number: 0050016432

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CHAPTER 1 - INTRODUCTION

1.1 - Introduction

The role of foundation in superstructure is undeniably important especially in high-rise building. From ancient periods, human had been already paid attention to the foundation structure. It can be evidenced in the bible.

I will show you what he is like who comes to me and hears my words and puts theme into practice. He is like a man building a house, who dug down deep and laid the foundation on rock. When a flood came, the torrent struck that house but could not shake it, because it was well built. But the one who hears my words and does not put them into practice is like a man who built a house on the ground without a foundation. The moment the torrent struck that house, it collapsed and its destruction was complete.

[Luke 6:47-49 NIV (Circa AD 60)]

The above statement is well proving that the significance of foundation. Figure 1-1 and Figure 1-2 are showing the results of superstructure building on the poor foundation.

Figure 1-1 - Pisa of Tower in Italy

Resource: http://www.flickr.com/photos/heute/337699799/

Figure 1-2 - Building collapse due to poor foundation, China (Jun-2009)

Resource: http://141hongkong.com/forum/redirect.php?tid=373616&goto=lastpost

Deep foundations have been frequently used to support structures, such as buildings, bridges, towers, and dams, in areas where the soil conditions are unfavorable for shallow foundations. Two basic types of deep foundation are well known as drilled shafts and driven piles. Drilled shafts are usually installed by using hammer to drive the steel or concrete piles into the ground. This report is concentrated on the improvement of design and quality control of driven piles.

Traditionally, the design of pile foundations is based on static analysis methods, such as α-method, β-method, λ-method, Nordlund's method, Meyerhof's method, SPT (Standard Penetration Test) method, and CPT (Cone Penetration Test) method. However, the pile capacity estimated from static analysis based on the soil parameters obtained from the laboratory test and in-situ test may not be accurate. It can vary widely due to original nature of soil deposit, and different soil testing and evaluation methods. Consequently, some type of load-carrying capacity method of driven piles is needed in the field.

Static load test can serve as the ultimate verification of driven pile capacity, though this method has still the unsettled question which concerns the definition of failure load. However, the static load test is expensive and time-consuming to carry out and thus can only be performed on a few selected driven piles. The information obtained from static load test results cannot be easily estimated to other piles at the site, leaving those other piles to be unverified. Besides, the information obtained from the static load tests cannot help engineer to decide when to stop pile driving. Estimation of pile capacity based on observation during pile driving is therefore an essential element in a pile driving practice.

With the creation of digital computers, synchronizing analysis of pile driving problem during pile driving became possible. In 1960, Smith proposed the first applicable wave equation method for pile-driving analysis. The wave equation method was adopted by the engineers in the piling industry because it gave a more accurate result. With the development and acceptance of PDA (Pile Driving Analyzer), the Case method (Rausche et al. 1985), applying the measured force and velocity in approximate simple algebraic equations, has become one of the widely used methods to evaluate the pile capacities during pile driving. However, the static soil resistances determined by the Case method are very sensitive to assumed Case damping factor, and shaft and toe resistance cannot be separated by the Case method.

In an effort to solve some of the weakness of the Case method, an alternative procedure - the CAse Pile Wave Analysis Program (CAPWAP), was developed by Rausche et al. (1972, 1985). The CAPWAP approach is based on the one-dimensional wave propagation model suggested by Smith (1960). In the CAPWAP analysis, the data (either force or velocity) is used as input to match closely with the other by adjusting Smith model parameters. Once the acceptable match is achieved, then the pile capacities and the Smith model parameters can be determined. In addition, some other researchers such as Paikowsky et al.

(1994), Hirsch et al. (1976), Liang and Zhou (1996), and Liang (2003) have proposed various methods to interpret the measured force and velocity for evaluating the pile capacity. However, the capacity estimation and drivability predictions from dynamic pile tests are sometimes far from satisfactory. Thus, the dynamic testing technique still needs to be further improved.

Nowadays, dynamic pile tests are widely adopted to verify when the design pile capacity is reached during pile driving and to monitor the installation process to avoid pile damage due to hammer impact. However, there is a lack of acceptance criteria for the number of pile tests and measured capacity. Hannigan et al. (1998) suggested that a minimum of two dynamic pile tests to be conducted for a small project. For large projects or small projects with expected installation difficulties, or significant time dependent capacity issues, a greater number of dynamic pile tests are recommended. If the test piles do not reach a required design capacity, the design load for the piles must be reduced or additional number of piles or longer pile length must be installed. The recommendations of the number of dynamic pile tests are totally based on engineering experience.

Paikowsky et al. (2004) provided recommendations for the number of piles to be dynamically tested as well as the acceptance criterion for a set of driven piles. The testing in manufacturing was employed to determine the number of dynamic tests to be performed on production piles. However, the adoption of normal distribution for dynamic test methods, which deviated from the actual cases that the probabilistic characteristics of dynamic test methods can be properly represented by the lognormal distribution, makes the recommendations very conservative. The acceptance criterion for a set of dynamic test results is subjectively chosen such that the average capacity of the tested piles is no less than 85% of the ultimate capacity. In addition, the application of arithmetic mean of measured capacity is far away from the actual cases of geometric mean of measured capacity when the lognormal distribution is used to represent the probabilistic characteristics of dynamic test methods. Therefore, there is a need to develop more reasonable, reliability-based quality control criteria for driven piles.

Since 1994, American Association of State Highway and Transportation Officials (AASHTO) has been in process to change from Allowable Stress Design (ASD) method to Load and Resistance Factor Design (LRFD) method for foundation design. Whereas ASD considers all uncertainties in the applied load and ultimate geotechnical or structural capacity in a single value of factor of safety (FS), LRFD separates the variability of these design components by applying load and resistance factors to the load and material capacity. Comparing to ASD, LRFD has the following advantages:

- Accounts for variability in both resistance and load;

- Achieves relatively uniform levels of safety based on the strength of soil and rock for different limit states and foundation types;

- Provides more consistent levels of safety in the superstructure and substructure as both are designed using the same loads for predicted or target probabilities of failure.

The adoption of LRFD approach makes possible the application of reliability analysis to quantify uncertainties joined to various methods for estimating loads and resistances. In AASHTO LRFD specifications (2003), the resistance factors for various design methods are recommended with calibrations mostly based on the reliability analysis using available statistical data. The design of foundation piles is usually performed with static analytical calculations using both of the soil parameters from local geotechnical site investigations and laboratory test results. The uncertainties related to the prediction method, the errors of calculation model, and the spatial variability of soil parameters, are considered in a single resistance factor recommended for a specific design method regardless of individual site-specific situation. For each construction site, soil profiles, soil types, pile driving equipment, and hammer performance, will be unique. Thus, it is advantageous if a site-specific calibration for the resistance factors can be performed to improve design.

With the use of both static load test and dynamic pile test, the pile length estimated using static analysis methods during the design stage would be proven either adequate or inadequate based on field pile tests during pile driving. Therefore, the uncertainties of static analysis design method could be reduced by dynamic test results. Vrouwenvelder (1992) presented an approach for including either the static or dynamic test results to update the factor of safety in the ASD method of driven piles. Zhang et al. (2002) demonstrated that the results from static pile load tests could be included into pile design using Bayesian theory by updating the resistance factor in LRFD. The practice in Ohio Department of Transportation shows that more and more dynamic pile test methods have been utilized to compare with static load test, due to saving in cost and time. Recognizing that dynamic pile testing is much preferred pile capacity verification method. A methodology needs to be developed to update the resistance factors for static analysis method by utilizing dynamic pile test results.

The fact is that axial capacity of a driven pile may change over time after initial pile installation has been reported by a number of geotechnical engineers for many years. The increase of pile capacity with time is usually referred to the soil set-up. Oppositely, the decrease of pile capacity with time is often named as soil relaxation. Due to pile driving, soils around the pile are disturbed and remolded, and excess pore pressures are generated. With passing of time, the excess pore pressure will dissipate and consequently pile capacity is built up. Decrease in excess pore pressure is inversely proportional to the square of the distance from the pile (Pestana et al. 2002). The time to dissipate excess pore pressure is proportional to the square of horizontal pile dimension (Holloway and Beddard

1995; Soderberg 1961), and inversely proportional to the soil's horizontal coefficient of consolidation (Soderberg 1961). Accordingly, larger-diameter piles take longer time to set-up than small-diameter piles (Long et al. 1999; Wang and Reese 1989). As excess pore pressures dissipate, the effective stress of the affected soil increases, and set-up predominately occurs as a result of increased shear strength and increased lateral stress against the pile. In experience, piles driven into clay tend to greater set-up than piles driven into sand and silt. Piles may be relaxation when driven into dense and saturated sand and silt. Based on observations in the field, numerous geotechnical engineers developed various empirical formulas to predict the set-up behavior (e.g., Skov and Denver 1988; Svinkin et al. 1994; Huang 1988; Zhu 1988). In particular, the semilogarithmic empirical relationship, proposed by Skov and Denver, has been widely used to predict post-installation pile capacity increase with time.

With an accumulation of more experience and knowledge on set-up phenomenon, some researchers have suggested that the set-up be formally incorporated into the prediction method to determine total pile capacity. For example, Bullock et al. (2005) proposed a conservative method for incorporating side shear set-up into the total pile capacity. The predicted set-up capacity was assumed to have the same degree of uncertainties as the measured reference capacity and a single safety factor was used to account for all uncertainties of loads and resistances. Due to different uncertainties associated with measured capacity and predicted set-up capacity, Komurka et al. (2005) proposed a method to apply separate safety factors to End of Driving (EOD) and set-up components of driven pile capacity. Furthermore, the set-up capacity was characterized as a function of pile penetration based on dynamic monitoring during both initial driving and restrike testing. The separate safety factors recommended for EOD and set-up capacity, however, are based purely on judgment with no attendant database and statistical analysis. Therefore, the development of a reliability analysis methodology on set-up capacity will be desirable to separate the resistance factors in LRFD of driven piles.

1.2 - Objectives

The objectives of this study are listed as follows:

- To present a methodology for pile design by combining the information from the static calculation and dynamic pile testing.

- To develop a one-dimensional wave equation based to interpret the High Strain Testing (HST) data for estimation of the shaft and toe resistance of driven piles.

- To develop a methodology for pile driving by monitoring the driving hammer energy transfer into the pile during driving. The results from dynamic loading test will present the different between steel piles and concrete pile.

- To find the cause of damaging pile during driving. Pile integrity are determined based on the obtaining the change of pile impedance.

- To develop the relationship between allowable pile stress, pile integrity and driving energy from the dynamic loading test result.

1.3 - Organization of report

The organization of this report is as follows:

- Chapter 2 presents the literature review of analysis methods of driven piles.

- Chapter 3 presents the dynamic formula.

- Chapter 4 presents the dynamic analysis by wave equation.

- Chapter 5 presents the dynamic pile testing and analysis.

- Chapter 6 presents the test results which are obtained from West Libya project and Macau project.

- Chapter 7 is the discussion and conclusion of the test results from Chapter 6.

CHAPTER 2 - LITERATURE REVIEW

2.1 - Introduction

Dynamic analysis methods can be defined as analytical techniques for evaluating the soil resistance when the pile is being driven. A pile foundation designed to meet compression, uplift, and lateral load performance requirements using the static design methods is adopted if it cannot be installed as designed and without damaged. The suitability of a selected pile section to be driven within allowable driving stress limits to require ultimate capacity and the minimum pile penetration depth should be evaluated by the foundation design.

The soil resistance acting on the pile during driving is including the static and dynamic resistance. The primary interest is the static resistance component because this is the only resistance available to support the 'future' designed loads. During driving, the static resistance is in most cases on a part of the ultimate pile capacity. The dynamic soil resistance, or damping force, is the temporary viscous resistance on the pile during driving. Therefore, the dynamic resistance provides resistance to the pile penetration during driving but does not provide long term support under static loading conditions.

Traditional dynamic analysis has been dynamic formulas such as the Engineering News formula. Depending on the formula used, an estimate of the allowable or ultimate pile capacity related to the pile driving resistance at the time of driving is obtained. Unfortunately, dynamic formulas have fundamental weaknesses in that they do not totally represent the dynamic force of the hammer-pile impact, the influence of axial pile stiffness and / or the soil behaviour. Dynamic formulas have also proven unreliable in determining pile capacity in many conditions. Their continued use is not recommended on significant projects.

Wave equation analysis, Goble and Rausche (1986), is the most readily available dynamic analysis tool to the foundation designer during the design stage. A detailed discussion of the wave equation method is presented in following Chapter 4. Dynamic testing and analysis, Goble and Hussein (1994), Hannigan (1990) is an additional dynamic analysis tool that can be used if a design stage test program is planned. Additional details on dynamic testing and analysis methods are presented in following Chapter 5.

These dynamic analysis methods not only provide an estimate of the ultimate pile capacity relative to pile driving resistance, also include an evaluation of actual pile driving stresses. The application of dynamic analysis method is to match the hammer size and pile section to the static and dynamic soil resistance. Moreover, it can find the ultimate pile capacity or to reach the specified pile penetration depth.

2.2 - Dynamic analysis methods

Piles are forced into the ground by dynamic means such as impact or vibration. A successful pile foundation which meets the design objectives depends on relating the static analysis results presented on the plans to the dynamic methods of field installation and control. During the design and construction stage, the following site specific questions often arise:

- Can the design pile section be driven to the required penetration depth and capacity with readily available pile hammers (design stage) or a proposed hammer (construction stage)?

- What soil resistance must be overcome? With the expected or proposed hammer, what will be the maximum driving resistance required to overcome this soil resistance and what will be the allowable stress limit by the design pile section during driving?

- If a specific hammer cannot drive the design pile section to the required depth and / or capacity within allowable driving stress, what hammer characteristics could be specified (design stage) or obtained (construction stage) to drive the pile?

To answer these and further questions that may arise with a specific pile foundation, analysis of the hammer-cushion-pile-soil system through dynamic analysis methods is invaluable. However, based on the past experience, is not sufficient to answer the above questions.

The traditional method for field verification of the pile capacity is dynamic formulas which are discussed in detail in Chapter 3. Unfortunately, dynamic formulas have fundamental weaknesses and cannot provide reliable answer of the above questions. Dynamic formulas do not provide allowable pile driving stresses and have proven unreliable in determining pile capacity in many conditions. Therefore, the dynamic analysis methods should be used in both the design and construction stages of a project. In a design stage, wave equation analysis may indicate whether the pile section cannot be driving stresses or within a reasonable driving resistance. A design amending shall then be considered. The wave equation can be used to evaluate what changes can be made i.e. pile size, pile type, pile material properties, hammer size, or what installation techniques can be specified to achieve the desired foundation. If a test pile program is preformed during the design stage, the information from dynamic testing and analysis of test piles in conjunction with wave equation analyses can be used to evaluate design change.

If a project is designed without dynamic analysis methods, and then problems are detected when these methods are executed during the construction stage, problem solutions may not be quite as easy. In this case, equipment and materials may already be on-site, thereby the solutions is limited. For example, few cost effective options exist once a thin walled pipe pile lacking the required drivability arrives on site. In this example, it may be necessary to reduce the ultimate capacity per pile and increase the number of piles. Moreover, it also uses a pile installation aid such as predrilling, or order new piling having the necessary drivability. Of course, it assumes that the hammer and crane are still suitably sized. While a construction stage problem is more complicated, dynamic analysis methods still offer the most reasonable way of determining the most cost effective solution.

2.3 - Methods of dynamic analysis

There are two methods of dynamic analysis. These include:

- Wave equation analysis
- Dynamic testing and analysis

The wave equation is a computer simulation of the pile driving process that models wave propagation through the hammer-pile-soil system. This computer analysis can be readily used in either the design or construction stage to evaluate pile drivability, size of driving equipment, calculate driving stresses, and assess ultimate pile capacity versus pile penetration resistance. These analyses are an important improvement over the use of dynamic formulas. Two limitations of wave equation analysis involve assumptions that must be made on drive system performance and on the soil model, (i.e., the soil resistance distribution, and the soil quake and damping parameters).

Dynamic testing and analysis consists of measuring strain and acceleration near the pile head during driving, or restriking using a Pile Driving Analyzer or similar data processing device conforming to ASTM D4945:2000. The strain and acceleration signals are used to calculate quantities such as energy transfer, pile driving stresses, and estimates of ultimate pile capacity. Further analysis of dynamic testing data using signal matching methods can also figure the soil model. The information from dynamic testing on drive system performance and the soil model can be used to improve the accuracy of wave equation results. Dynamic testing and analysis provides a better evaluation method and construction control as compared to dynamic formulas.

2.4 - Driving resistance criteria

The foundation designer shall specify the dynamic analysis method to be used for determining of the driving resistance. The driving resistance usually includes of a specified penetration resistance at a given hammer stroke and a minimum pile penetration depth.

In the past, dynamic formulas were the primary means of establishing the driving resistance criteria. As discussed, dynamic formulas do not provide information on pile driving stresses and have proven unreliable in determining pile capacity in many conditions. Therefore, it is not recommended to be adopted continually on significant projects.

The wave equation analysis offers a rational means of establishing a relationship between the static pile capacity of a driven pile with the number of blows per 250 millimeter required by a particular hammer to drive a selected pile to an ultimate capacity in a given soil situation. The driving criteria established from wave equation analysis should be substantiated by static load tests whenever possible.

Dynamic testing and analysis of indicator or test piles allows an assessment of the static pile capacity during driving. This is also an appropriate means of establishing driving criteria. Again, the driving criteria established by dynamic testing and analysis should be proven by static load tests whenever possible.

Driving criteria shall also consider time dependent changes in pile capacity. Hence, lower driving resistances than required may be acceptable in soils where soil setup is expected. When there are higher driving resistances, soil relaxation is not anticipated. Once again the driving criteria should be substantiated by static load tests whenever possible. In cases where time dependent soil strength changes are expected, load tests should be delayed an appropriate waiting period until the expected soil strength changes have occurred.

2.5 - Quality Control on Driven Piles

The most common static analysis methods used for evaluating the static axial capacity of driven piles are as follows: α-method (Tomlinson, 1986), β-method (Esrig & Kirby, 1979), λ-method (Vijayvergiya and Focht, 1972), Nordlund's method (Nordlund, 1963), Nottingham and Schmertmann's CPT method (Nottingham and Schmertmann, 1975), and Meyerhof's SPT method (Meyerhof, 1976). Nordlund's method, β-method, Nottingham and Schmertmann's CPT method, and Meyerhof's SPT method are generally used when calculating the design capacity of driven piles in cohesionless soils, while α-method, β- method, λ-method, and Nottingham and Schmertmann's CPT method are used to predict the pile capacity when piles are driven into cohesive soils. However, the pile capacity estimated from static analysis based on the soil parameters obtained from the laboratory test and in-situ test may not be accurate. Thus, static load test, dynamic pile test, or both which are believed to have higher accuracy in estimation of pile capacity have been performed to verify the design capacity calculated from static analysis method.

The static load tests have been performed to verify that the behavior of the driven piles agreed with the assumption of the design for decades. There are various definitions of pile capacity evaluated from load-movement records of a static load test. Four of them are of particular interests; namely, the Davisson Offset Limit (Davisson 1972), the DeBeer Yield Limit (DeBeer 1968), the Hansen Ultimate Load (Hansen 1963), and Decourt extrapolation (Decourt 1999). NCHRP Report 507 (Paikowsky et al. 2004) presented that Davisson's Pile failure criterion could be used to determine the pile capacity for driven piles, irrespective of the pile diameter and the static load test procedure. The static load tests have been accepted by most geotechnical engineers as the most accurate evaluation method of pile capacity. However, the cost and time needed for a static load test hindered its extensive application in field testing. Dynamic pile testing, an alternative approach of verification of pile design capacity in the field, has become more and more attractive in geotechnical engineering due to its savings in cost and time.

The past one hundred years or longer, many attempts have been made to predict the driving characteristics and the bearing capacity of piles through the use of dynamic energy formulas. Dynamic energy formulas are based on simple energy balance relationship which input energy is equal to the sum of consumed energy and lost energy. In USA, most of the state highway departments still widely use the Engineering News Record (ENR) formula (Wellington, 1892) and its modified version for estimating pile capacities. However, there are some reasons that make these formulas less satisfactory:

1 - Rigid pile assumption;

2 - No consideration of soil-pile interaction;

3 - No incorporation of damping factor.

Although many efforts have been made to improve the dynamic energy formulas (Gates 1957, Liang and Husein 1993, Paikowsky et al. 1994, Liang and Zhou 1996), the accuracy of estimation from dynamic energy methods is still far from satisfactory.

As a better alternative to energy formulas for pile driving, the wave equation method was proposed by Smith (1960) for the first practical use in estimating the pile capacity. The one-dimensional wave equation was derived to describe the motion of pile particles by applying Newton's Second Law to a differential element of an elastic rod. In Smith model, the pile is separated into lumped masses and connected by pile "spring". The soil resistance to driving is provided by a series of springs that are assumed to behave in a perfectly elastic-plastic manner, and the spring stiffness is defined by the ratio of the maximum static resistance of the soil R_s and the maximum elastic deformation or quake. Damping coefficients are introduced to account for the viscous behavior of the soil. The total soil resistance R_t is given by

$$R_t = R_s(1 + JV) \tag{2-1}$$

where R_s = the static soil resistance

J = the Smith damping coefficient

V = pile velocity

In 1975 and 1976, the other two of the soil models were proposed. They are Case model and TTI model, by Goble at Case Western Reserve University and Hirsch at Texas Transportation Institute. These two models can be viewed as modified version of Smith model. In Case model, the soil damping force is uncoupled from the spring force and is dependent on the pile particle velocity. Case damping factor J_c was introduced to account for the soil damping effect when multiplied with pile impendence Z_p and toe velocity V. The total resistance R_t is given by

$$R_t = R_s + J_c Z_p V \tag{2-2}$$

where Z_p = Pile impendence

J_c = the damping coefficient

V = pile toe velocity

R_s = the static soil resistance

It is worth noting that J_c is not related to soil properties, but a pure empirical value calibrated with the results of static load tests.

In TTI model, the nonlinearity of soil damping force with velocity is taken into account. The total resistance R_t during driving is given by

$$R_t = R_s + (JV^N) \tag{2-3}$$

where N = an exponent less than unity to reflect the nonlinearity of the damping force with velocity

J = the damping coefficient

V = pile velocity

R_s = the static soil resistance

An exponent less than unity to reflect the nonlinearity of the damping force with velocity; 0.2 can be taken for N when lack of information.

With the development and acceptance of pile driving analyzer (PDA), Case method has become one of the widely used methods to evaluate the pile load capacity. However, its inaccuracy has also been reported (Lai and Kuo 1994; Paikowsky et al. 1994). The static soil resistances determined by the Case method are very sensitive to the assumed Case damping factor and the shaft and the toe resistance cannot be separated by the Case method. In an effort to overcome these shortcomings, Rausche (1972, 1985) developed an alternative procedure known as the CAse Pile Wave Analysis Program (CAPWAP). The CAPWAP is also based on the one-dimensional wave propagation model suggested by Smith (1960). In CAPWAP analysis, the High Strain Test (HST) data on the pile head (either force or velocity) is used as input to generate output that would match closely with the other HST data by adjusting Smith model parameters. The acceptable match is achieved and then the pile capacities and the Smith model parameters can be determined. However, the numerical procedure in CAPWAP (lumped mass and springs) is computationally time-consuming, and the capacity estimation and drivability predictions from dynamic pile tests are sometimes far from satisfactory. Thus the dynamic testing technique still needs to be further improved.

Nowadays, dynamic pile tests are widely used to verify with the design pile capacity during pile driving and to monitor the installation process for avoiding pile damage due to hammer impact. Hannigan et al. (1998) presented that the number of piles that should be dynamically tested on a project depends on the project size, variability of the subsurface conditions, the availability of static load test information and the reasons for performing the dynamic tests. A minimum of two dynamic pile tests is recommended to be conducted for a small project. For large projects or small projects with expected installation difficulties or significant time dependent capacity issues, a larger number of dynamic pile tests are recommended to be executed. On larger projects, CAPWAP analyses are typically performed on 20 to 40% of the dynamic test data obtained from both initial driving and restrike dynamic tests. If the test piles do not achieve a prescribed design capacity, the design load for the piles must be reduced or additional piles or pile lengths must be installed. The recommendations of the number of dynamic pile tests are totally based on engineering experience.

2.6 - Methods for determining pile drivability

There are three available methods for predicting and/or checking pile drivability. All of the methods have advantages and disadvantages and are therefore presented in order of increasing cost and reliability.

2.6.1 - Wave Equation Analysis

This method accounts for the pile impedance and predicts driving stresses as well as the relationship of the pile driving resistance versus ultimate pile capacity. Wave equation analysis preformed in the design stage requires assumptions on the hammer type and performance level, the drive system components, as well as the soil response during driving. These shortcomings are reflected in variations between predicted and actual field. Even these shortcomings, the wave equation is a powerful design tool that can use to check drivability in the design stage, to design an appropriate pile section, or to specify driving equipment characteristics.

2.6.2 - Dynamic testing analysis

Dynamic testing can be carried out during pile installation to calculate during stresses and to estimate static pile capacity at the time of driving. Time dependent changes in pile capacity can be evaluated if measurements are made during restrike tests. Additional signal matching analysis can also provide soil parameters for further wave equation analysis. A shortcoming of this method as a design tool is that it must be performed during pile driving. Therefore, in order to use dynamic testing information to confirm drivability or to refine a design, a test program is required during the design stage.

2.6.3 - Static load tests

Static load tests are useful for checking drivability and confirming pile capacity before the production pile driving. Test piles are normally driven to estimated lengths and load tested. The confirmation of pile drivability through static load testing is the most accurate method of confirming drivability and pile capacity since a pile is actually driven and load tested. However, this advantage also illustrates its shortcoming for determining drivability includes:

- Cost and time delay that limit their suitability to certain projects.

- Assessment of driving stresses and pile damage are not provided by the test.

- Can be misleading on projects where soil conditions are highly variable.

As design and construction control tools, method 1 and 2 offer additional information and complement static load tests. Method 1 and 2 can save the material costs or reduction of construction delays. These methods can also be used to reduce the number of static load tests and also allow evaluation of increases in the maximum allowable design stresses. A determination of the increase (soil setup) or decrease (relaxation) in pile capacity with time can also be made if pile is restriked after initial driving.

2.7 - Drivability versus pile type

Drivability shall be checked during the design stage of all driven piles. It is important for closed end steel pipe piles where the impedance of the steel casing may limit pile drivability. Although the designer may use to specify a thin-wall pipe in order to save material cost, a thin wall pile may lack the drivability to develop the required ultimate capacity or to achieve the necessary pile penetration depth. Wave equation analyses shall be performed in the design stage to select the pile section and wall thickness.

Steel H-piles and open pipe piles, prestressed concrete piles, and timber piles are also subject to drivability limitations. This is true as allowable design stresses increase and special design events require increased pile penetration depths. The drivability of long prestressed concrete piles can be limited by the pile’s tensile strength.

CHAPTER 3 - DYNAMIC FORMULAS

3.1 - Introduction

Since engineers began using piles to support structures, they have attempted to find rational methods for determining the pile's load carrying capacity. Methods for predicting capacities were proposed, using pile penetration observations obtained during driving. The realistic measurement that could be obtained during driving was the pile set per blow. Therefore energy concepts applying the kinetic energy of the hammer to resistance on the pile as it penetrates the soil were developed. The equation can be expressed as:

$$Wh = Rs_b \tag{3-1}$$

where W = Ram weight

h = Ram stroke

R = Soil resistance

s_b = Set per blow

This type of expression is known as dynamic formulas. Because of the simplicity, dynamic formulas have been widely used for many years. More comprehensive dynamic formulas include consideration of pile weight, energy losses and other factors in drive system components. Whether simple or more comprehensive dynamic formulas are used, pile capacities determined from dynamic formulas have shown poor correlations and wide scatter when statistically compared with static load test result. Therefore, except well supported empirical correlations under a given set of physical and geological conditions are available, dynamic formulas should not be used.

3.2 - Accuracy of dynamic formulas

Wellingtion proposed the popular Engineering News formula in 1983. It was developed for evaluating the capacity of timber piles driven primary with drop hammers in sands. Concrete and steel piles were unknown at that time as were many of the pile hammer types and sizes used today. Therefore, it should be little surprise that the formula performs poorly in predicted capacities of pile foundations.

The inadequacies of dynamic formulas have been known for a long time. In 1941, an ASC committee on pile foundations assembled the results of numerous pile load tests along with the predicted capacities from several dynamic formulas, including the Engineering News, Hiley, and Pacific Coast formulas. The mean failure load of the load test data base was 91 tons. After reviewing the data base, Peck (1942) proposed that a new and simple dynamic formula could be used that stated the capacity of every pile was 91 tons. Peck concluded that the use of this new formula would result in a prediction statistically closer to the actual pile capacity than obtained by using any of the dynamic formulas contained in the 1941 study.

More recently, Chellis (1961) noted that the actual factor of safety obtained by using the Engineering News formula varied from as low as 2/3 to as high as 16. Sowers (1979) reported that the safety factor from the Engineering News formula varied from as low as 2/3 to as high as 20. Fragasny et al. (1988) in the Washington State DOT study entitled “Comparison of Methods for Estimating Pile Capacity” found that Hiley, Gates, Janbu, and Pacific Coast Uniform Building code formulas all provide relatively more dependable results than the Engineering News formula.

3.3 - Problems with dynamic formulas

Dynamic formulas are fundamentally incorrect. The problems complied with pile driving formulas can be traced to the modeling of each component within the pile driving process. They are the driving system, the soil, and the pile. Dynamic formulas offer a poor representation of the driving system and the energy losses of drive system components. Dynamic formulas also assume a rigid pile, thus resistance is constant and instantaneous to the impact force. A more detailed discussion of these problems is presented below.

First, the derivation of most formulas is not based on a realistic treatment of the driving system. Most formulas only consider the kinetic energy of the driving system. The variability of equipment performance is typically not considered. Driving systems include many elements in addition to the ram, such as the anvil for a diesel hammer, the helmet, the hammer cushion, and for a concrete pile, the pile cushion. These components affect the distribution of the hammer energy with time, both at end after impact, which influences the magnitude of force. The force determines the ability of the driving system to the pile into the soil.

Second, the soil resistance is assumed untreated that it is a constant force. This assumption neglects the most obvious characteristics of real soil behavior. The dynamic soil resistance is the resistance of the soil to pile penetration produced by hammer blow. This resistance is not equal to the static soil resistance. However, most dynamic formulas consider the resistance during driving equal to the static resistance or pile capacity. The penetration of the pile into the soil during driving is resisted not only by static friction and cohesion. It is also by the soil viscosity which is comparable to the viscous resistance of liquids process creates dynamic resistance force along the pile shaft and at the pile toe. The soil resistance during driving is not equal to the static soil resistance or pile capacity under static loads.

Third, the pile is assumed to be rigid and its length is not considered. This assumption completely neglects the pile’s flexibility which affects its ability to penetrate the soil. The energy delivered by the hammer sets up time-dependent stresses and displacements in the helmet, in the pile, and in the surrounding soil. In addition, the pile behaves, not as a concentrated mass, but a long elastic rod in which stresses travel longitudinally as waves. Compressive waves which travel to the pile toe are responsible for advancing the pile into ground.

3.4 - Dynamic formulas

As noted in previous section, the Engineering News formulas is generally recognized to be one of the least accurate and least consistent of dynamic formulas. Due to the overall poor correlations documented between pile capacities determined from this method and static load test results, the use of the Engineering News formula is not recommended.

For small projects where a dynamic formula is used, statistics indicate that the Gates formula is preferable, since it correlates better with static load test results. The gates formula presented below has been revised to reflect the ultimate pile capacity in kilonewtons and includes the 80 percent efficiency factor on the rated energy, E_r, recommended by Gates.

$$R_u = [7\sqrt{E_r}\log(10N_b)] - 550 \tag{3-2}$$

where R_u = The ultimate pile capacity [kN]

E_r = The manufacturer's rated hammer energy (Joules) at the field observed ram stroke

$Log(10N_b)$ = Logarithm to the base 10 of the quantity 10 multiplied by N_b, the number of hammer blows per 25mm at final penetration

It is desirable to calculate the number of hammer blows per 0.25 meter of pile penetration, N_{qm}, required obtaining the ultimate pile capacity. The gates formula can be written in the following form:

$$N_{qm} = 10(10^x) \tag{3-3}$$

where $x = \left[\frac{(R_u + 550)}{7\sqrt{E_r}}\right] - 1$

Most dynamic formulas are in terms of ultimate pile capacity, rather than allowable or design load. For ultimate pile capacity formulas, the design load shall be multiplied by a factor of safety to obtain the ultimate pile capacity that is input into the formula to determine the "set", or amount of pile penetration per below required. A factor of safety of 3.5 is recommended when using the gates formula. For example, if a design load of 700kN is required in the bearing layer, then an ultimate pile capacity of 2450kN should be used in the Gates formula to determine the necessary driving resistance.

3.5 - Alternatives to use of Dynamic formulas

Most shortcomings of dynamic formulas can be overcome by a more realistic analysis of the pile driving process. The one-dimensional wave equation analysis discussed in Chapter 4 is a more realistic method. However wave equation analyses are primarily preformed on main frame computers. Therefore, wave equation analysis is often viewed as a tool for special projects and is not widely used. With the widespread use of computers in present, wave equation analysis can now be easily performed in a relatively short amount of time.

Dynamic methods of wave equation analysis, such as dynamic testing and analysis, are instead of traditional dynamic formulas. Dynamic methods shall be used in conjunction with static pile load tests and the use of dynamic formulas shall be discontinued.

CHAPTER 4 - DYNAMIC ANALYSIS BY WAVE EQUATION

4.1 - Introduction

As discussed in previous chapters, dynamic formulas with observed driving resistances do not produce acceptable accurate predictions of actual pile capacities. Furthermore, they do not provide information on stresses in the pile during driving. The wave equation analysis of pile driving can estimate many shortcomings associated with dynamic formulas by simulating the hammer impacts and pile penetration process. The term of wave equation refers to a partial differential equation. However it means a complete approach to the mathematical representation of a system consisting of hammer, cushions, helmet, pile and soil. It is also an associated with computer program for the convenient calculation of the motions and forces in this system after ram impact.

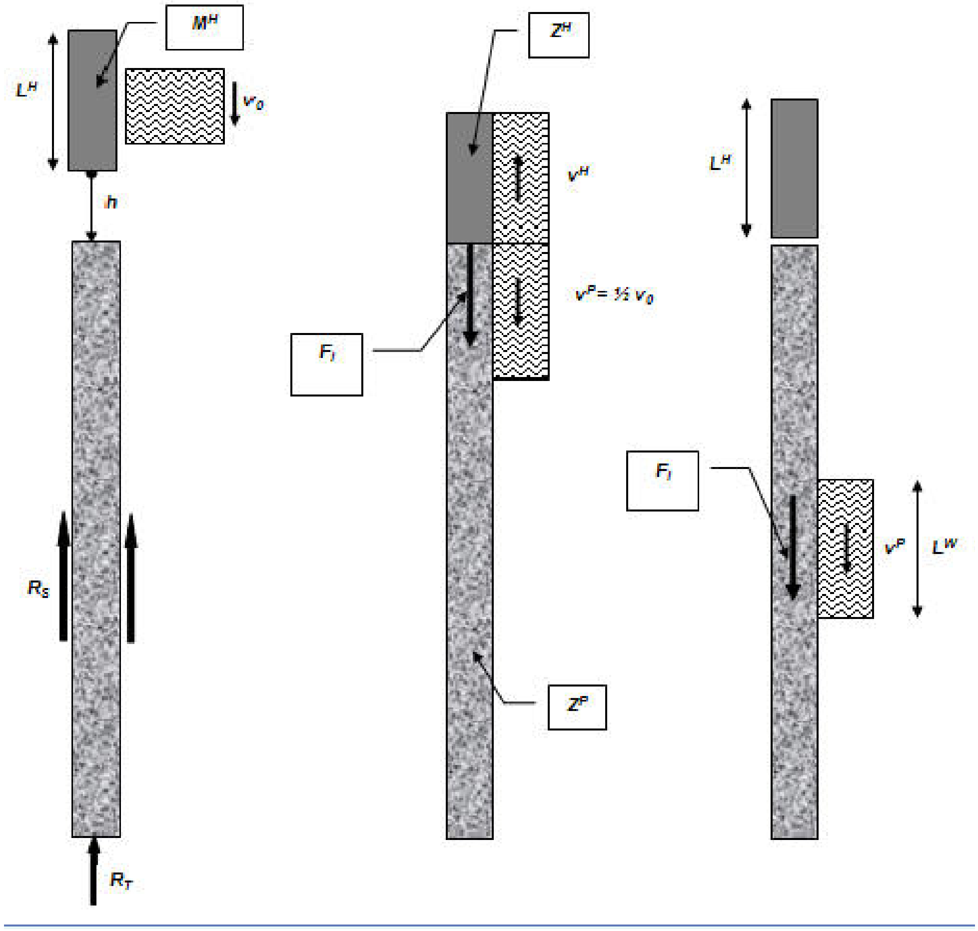
The approach was developed by E. A. L. Smith (1960), and after the rationality of the approach had been recognized, several researchers developed a number of computer programs. For example, the Texas Development of Highways supported research at the Texas Transportation Institute (TTI) in an attempt to reduce concrete pile damage using a realistic analysis method. FHWA sponsored the development of both the TTI program (Hirsch et al. 1976) and the WEAP program (Goble and Rausche, 1976). FHWA supported the WEAP development to obtain analysis results backed by measurements taken on construction piles during installation for a variety of hammer models. The WEAP program was updated several times under FHWA sponsorship, the last time (Goble and Rausche, 1986) when the WEAP86 program was released. Later, improved data files, refined mathematical representations and modernized user conveniences were added to this program on a proprietary basis, and the program is now known as GRLWEAP (Goble Rausche Likins and Associated, Inc., 1996). GRLWEAP is accepted for use on public projects by a variety of agencies (e.g. AASHTO, 1992, US Army Corps of Engineers, 1993), State Developments of Transportation, and the FHWA for routine analyses. However, this shall not be explained as a promotion or endorsement.

This chapter will explain what a wave equation analysis is, how it works, and what problems it can solve. Highlighting program applications will be demonstrated. Also, basic program usage and application of program results will be resented.

4.2 - Wave propagation

Input preparation for wave equation analyses is often very simple, requiring only very basic driving system and pile parameters in addition to a few standard soil properties. Thus, a wave equation program can be run without much specialized knowledge. However, interpretation of calculated results is assisted, and errors in result application may be avoided, by knowledge of the mechanics of stress wave propagation.

In the first moment, it is only compressed at the ram-pile interface when a pile is struck by hammer. This compressed zone, or force pulse, as shown in Figure 4-1, expands into the pile toward the pile toe at a constant wave speed, C, which depends on the pile's elastic modulus and mass density (or specific weight). When the force pulse reaches the embedded portion of the pile, its amplitude is reduced by the action of static and dynamic soil resistance forces. Depending on the magnitude of the soil resistances along the pile shaft and at the pile toe, the force pulse will generate either a tensile or a compressive force pulse which travels back to the pile head. Both incident and reflected force pulses will cause a pile toe motion and produce a permanent pile set if their combined energy and force are sufficient to overcome the static and dynamic resistance effects of the soil.



(a) (b) (c)

Figure 4-1 – Definition of parameters governing stress wave propagation in piles

Resource: K. Rainer Massarsch, (2008), Ground Vibrations Induced by Impact Pile Driving, Case Histories in Geotechnical Engineering, Arlington, VA

4.3 - Wave equation methodology

In a wave equation analysis, the hammer, helmet, and pile are modeled by a series of segments each consisting of a concentrated mass and a weightless spring. The hammer and pile segments are approximately one meter in length. Shorter segments often improve the accuracy of the numerical solution at the expense of longer times to generate the result. Spring stiffnesses are calculated from the cross sectional area and modulus of elasticity of the corresponding to pile section. Hammer and pile cushions are represented by additional springs which the stiffnesses are calculated from area, modulus of elasticity, and thickness of the cushion materials. In addition, coefficients of restitution (COR) are usually specified to model energy losses in cushion materials, and in all segments which can separate from their neighboring segments by a certain distance. The COR is equal to an elastic collision which preserves all energy and is equal to zero for a plastic condition which loses all deformation energy. Partially elastic collisions are modeled with an intermediate COR value.

The soil resistance along the embedded portion of the pile and at the pile toe is represented by both static and dynamic components. Therefore, both a static and a dynamic soil resistance force acts on every embedded pile segment. The static soil resistance forces are modeled by elasto-plastic springs and the dynamic soil resistance by linear viscous dashpots. The displacement at which the soil changes from elastic to plastic behavior is referred to as the soil “quake”. In the smith damping model, the dynamic soil resistance is proportional to a damping factor times the pile velocity times the assigned static soil resistance. A schematic of the wave equation hammer-pile-soil model is presented in Figure 4-2.

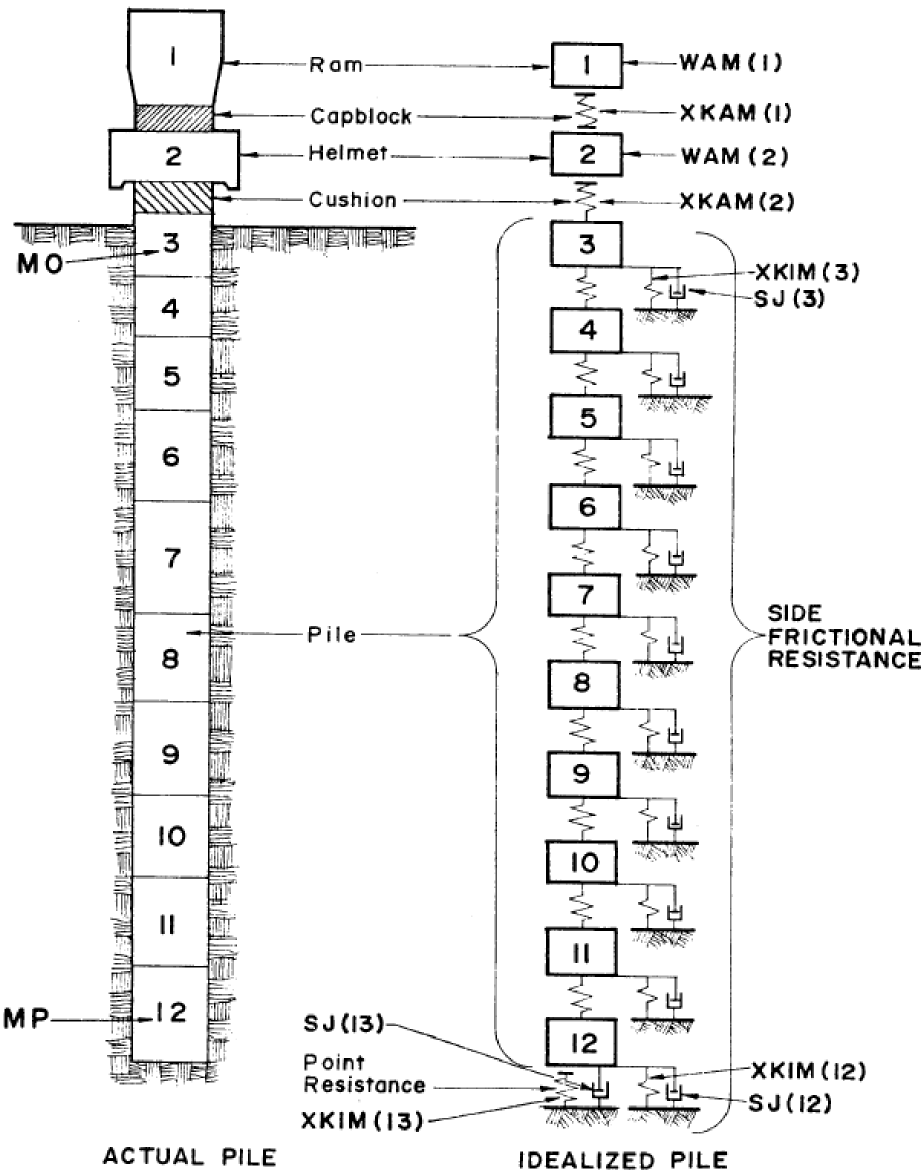


Figure 4-2 – Typical Wave Equation Model

Resource: Dr. Lee L. Lowery., P.E. Pile Driving Analysis By the Wave Equation, Department of Civil Engineering Texas A & M University College Station, Texas

As the analysis commences, a calculated or assumed ultimate capacity, R_{ut}, from user specified values is distributed according to user input among the elasto-plastic springs along the shaft and toe. Similarly, user specified damping factors are assigned to shaft and toe to represent the dynamic soil resistance. The analysis then proceeds by calculating a ram velocity using the input hammer efficiency and stroke. The ram movement causes displacements of helmet and pile head springs, and therefore compressions and related forces acting at the top and bottom of the segments. Furthermore, the movement of a pile segment causes soil resistance forces. A summation of all forces acting on a segment, divided by its mass, produces the acceleration of the segment. The product of acceleration and time summed over time is the segment velocity. The velocity multiplied by the time step generates a change of segment displacement which results in new spring forces. These forces divided by the pile cross sectional area at the corresponding section equal the stress at that point.

Similar calculations are made for each segment until the accelerations, velocities and displacements of all segments have been calculated during the time step. The analysis then repeats for the next time step using the updated motion of the segments from the previous time step. From this process, the accelerations, velocities, displacements, forces and stresses of each segment are computed over time. Additional time steps are analyzed until the pile toe begins to rebound.

The permanent set in millimeter unit of the pile toe is calculated by subtracting a weighted average of the shaft and toe quakes from the maximum pile toe displacement. The inverse of the permanent set is the driving resistance (blow count) in blows per meter that corresponds to the input ultimate capacity. By performing wave equation analyses over a wide range of ultimate capacities, a curve or “bearing graph” can be plotted which relates ultimate capacity to driving resistance.

A wave equation bearing graph is basically different from a similar graph generated from a dynamic formula. The wave equation bearing graph is associated with a single driving system, hammer stroke, pile type, soil profile, and a particular pile length. If any one of the above items is changed, the bearing graph will also change. Furthermore, wave equation bearing graphs also include the maxima of calculated compression and tension stresses.

4.4 - Wave equation applications

A bearing graph provides the wave equation analyst with two types of information:

- It establishes a relationship between ultimate capacity and driving resistance. From the user’s input data on the shaft and toe bearing resistances, the analysis estimates the permanent set (mm/blow) under one hammer blow. Specifying up to ten ultimate capacity values provides a relationship between ultimate capacity and driving resistance (or blow count) in blows per meter.

- The analysis also relates driving stresses in the pile to pile driving resistance.

The user usually develops a bearing graph or an inspector's chart for different pile lengths and uses these graphs in the field, with the observed driving resistance, to determine when the pile has been driven sufficiently for the required bearing capacity.

In the design stage, the foundation engineer should select typical pile types and driving equipment known to be locally available. Then by applying the wave equation analysis with various equipment and pile size combinations, it becomes possible to rationally:

- Design the pile section for drivability to the required depth and/or capacity.

  For example, considerations of soft layers may make it necessary to drive a pile through hard layers which driving resistance exceeds the resistance expected at final penetration. A thin walled pipe pile may have been initially chosen during design. However when this section is checked for drivability, the wave equation analysis may indicate that even the largest hammers will not be able to drive the pipe pile to the required depth because it is too flexible (its impedance is too low). Therefore, a wall thickness greater than necessary to carry the design load has to be chosen for drivability considerations.

- Assist in the selection of pile materials properties to be specified based on available driving stresses in reaching penetration and/or capacity requirements.

In the above example, it is supposed that it will be possible to drive the thinner walled pile to the desired depth but with excessive driving stresses. More buffers or reducing hammer energy will lower the stresses but will result in a refusal driving resistance. Choosing a high strength of steel grade can solve this problem. For concrete piles, higher concrete strength and/or higher prestress levels may provide acceptable solutions.

- Support the decision for a new penetration depth, design load, and/or different number of piles.

In the above example, after it has been determined that the pile section or its material strength had to be increased to satisfy pile penetration requirements, it may have become available to increase the design load of each pile and to reduce the total number of piles. Obviously, these considerations will require reviewing geotechnical and/or structural considerations.

Once the project has reached the construction stage, additional wave equation analyses shall be performed on the actual driving equipment by:

- Construction engineers – for hammer approval and cushion design.

  The pile type, material, and pile penetration requirements have been selected by the foundation designer, the hammer size and hammer type may have a decisive influence on driving stresses. For example, a hammer with adjustable stroke or fuel pump setting may have the ability to drive a concrete pile through a hard layer while allowing for reduced stroke heights and tension stress control when penetrating soft soil layers.

  Cushions are often chosen to reduce driving stresses. However, softer cushions absorb and distribute larger amounts of energy and thus increasing the driving resistance. Since it is both safer (reducing fatigue effects) and more economical to limit the number of blows applied to a pile. Softer cushions cannot always be chosen to maintain acceptable driving stresses. Also, experience has shown that changes of hammer cushion material are relatively ineffective for limiting driving stresses.

- Contractors – to select an economical combination of driving equipment to minimize installation cost.

While the construction engineer is interested in the safest installation method, constructors will like to optimize driving time for cost considerations. Fast hitting, light weight and simple hammers which have a high blow rate are obviously preferred. The wave equation analysis can be used to roughly estimate the expected number of hammer blows and the time of driving. This information is especially useful for a evaluation of the economy of driving systems.

Additional considerations may include the cost of pile cushions which are usually discarded after a pile has been installed. Thus, thick plywood pile cushions may be expensive.

4.5 - Analysis decisions for wave equation problems

4.5.1 - Selecting the proper approach

Even though the wave equation analysis is an invaluable tool for the pile design process, it shall not be confused with a static geotechnical analysis. The wave equation does not determine the capacity of a pile based on soil boring data. The wave equation calculates a driving resistance for an assumed ultimate capacity, or conversely. It assigns an estimated ultimate capacity to a pile based on a field observed driving resistance. It is one thing to perform a wave equation bearing graph for a certain capacity and a totally different matter to actually realize the capacity at a certain depth. The greatest disappointments happen when pile lengths required during construction vary significantly from those computed during design. To avoid such disappointments, it is imperative that a static analysis precede the wave equation analysis. The static analysis will yield an approximate pile penetration for a desired capacity or a capacity for a certain depth. The static analysis can also generate a plot of estimated pile capacity as a function of depth. It is important that the static analysis evaluates the soil resistance in the driving situation (e.g. remolded soil strengths, before excavation, before scour, before fill placement, etc.).

After the static analysis has been completed, a wave equation analysis can be performed either to a bearing graph or to driving resistances and stresses versus depth (drivability). Sometimes both analyses are performed. The bearing graph analysis is only valid within the approximate of analyzed soil profile depending on the variability of the soil properties. The drivability analysis calculates driving resistances and stresses for a number of penetration depths and therefore provides a more complete result. There is, however, a very basic difference between these two approaches. The bearing graph approach allows the engineer to assess pile capacity given a driving resistance at a certain depth. The drivability analysis points out certain problems that may occur during driving. If the pile actually drives differently from the wave equation predictions, then a reanalysis with different soil resistance parameters is needed to match the observed behavior.

Even though an accurate static analysis and a wave equation analysis have been performed with realistic soil parameters, the experienced foundation engineer will not be surprised if the driving resistance during pile installation is to differ basically from the prediction. Most likely the observed driving resistance will be lower than calculated. For example, a 500kN pile is driven into a clay. With a factor of safety of 2.5, the required ultimate capacity will be 1250kN. The static soil analysis indicates that the penetration of pile is 25m long to achieve for the ultimate capacity. There will ignore toe resistance and will base on remolded soil strength parameters. The soil may show only 50% of its long term strength during driving. It is therefore only necessary to drive the pile to a capacity of 625kN which shall be achieved at the 25 m depth. The expected end of installation driving resistance will correspond to 625kN. In a restrike test, say 7 days after installation, the 1250kN capacity will be expected and therefore a much higher driving resistance will be encountered than observed at the end of driving.

The above discussion points out one major reason for differences between analysis and reality. However, as with all mathematical simulations of complex situations, agreement of wave equation results with actual pile performance depends on the realism of the method itself and on the accuracy of the model parameters. The accuracy of the wave equation analysis will be poor when either soil model or soil parameters do not represent the state of maintenance of hammer or cushions. The pile behavior is satisfactorily represented by the wave equation approach in the majority of cases. A review of potential wave equation error sources follows.

4.5.2 - Hammer data input, external combustion hammers

The most important input quantity is the hammer efficiency. It is defined that the potential ram energy is available in the form of kinetic ram energy immediately during the time of impact. Many sources of energy loss are usually lumped into this one number. If the hammer efficiency is too high, then an optimistically low driving resistance will be predicted. This can lead to over predictions of ultimate pile capacity. If the efficiency is very low, the stresses may be under predicted, leading to possible pile failures during installation.

Hammer efficiency shall be reduced for battered pile driving. The efficiency reduction depends on the hammer type and batter angle. For hammers with internal ram energy measurements, no reductions are required. Hydraulic hammers often allow for a continuously adjustable ram kinetic energy which is measured and displayed on the control panel. In this case the hammer efficiency does not have to cover friction losses of the descending ram, but only losses that occur during the impact (e.g. due to improper ram-pile alignment) and it may therefore be relatively high. For such hammers, the wave equation analysis can select the proper energy level for control of driving stresses and economical driving resistances by trying various energy values in term of stroke which are lower than the rated value.

Similarly, air/stream hammers can be fitted with equipment that allows for variable strokes. The wave equation analysis can help to find that driving resistance which the stroke can be safely increased to maximum. It is, however, important to understand that the reduced stroke is often exceeded and the maximum stroke not fully reached. Corresponding increases and decreases of efficiency for the low and high stroke may therefore be necessary.

4.5.3 - Hammer data input, diesel hammers

The strokes of diesel hammer increases when the soil resistance and driving resistance increase. GRLWEAP simulates this behavior by trying a down stroke and repeats the analysis with the new value for the down stroke. The accuracy of the resulting stroke is therefore dependent on the realism of the complete hammer-pile-soil model and shall be checked in the field by comparison with the actual stroke. The consequences of an inaccurate stroke can be varied. For example, an optimistic assumption of combustion pressure can lead to high stroke predictions and therefore to non-conservative predictions of ultimate pile capacity while stress estimates wwill be conservatively high (which may lead to a hammer rejection).

Stroke and energy transferred into the pile appear to be closely related and large differences (say more than 10%) between stroke predictions and observations shall be explained. Unfortunately, higher strokes do not always mean higher transferred energy values. When a hammer pre-ignites, probably because of poor maintenance, the gases combusting before impact slow the speed of descending ram and cushion its impact. As a result, only a small part of the ram energy is transferred to the pile. A large part of the ram energy remains in the hammer producing a high stroke. In this case, the combustion pressure will be calculated by matching the computed with the observed stroke under the assumption of a normally performing hammer and then the calculated transferred energy will be much higher than the measured one and calculated blow counts will be non-conservatively low. It is therefore recommended that hammer problems are corrected as soon as possible on the construction site. If this is not possible then several diesel stroke or pressure options shall be tried when matching analysis with field observation and the most conservative results shall be selected.

4.5.4 - Cushion Input

Cushions are subjected to destructive stresses during their service and therefore its properties are changed continuously. Pile cushions experience a particularly pronounced increase in their stiffness because they are generally made of soft wood with its grain perpendicular to the load. Typically, the effectiveness of wood cushions in transferring energy increases until they start to burn. After that, they quickly deteriorate, this happens after approximately 1500 blows. To be conservative, the harder cushion (increased elastic modulus, reduced thickness) shall be used for driving stress evaluations and the less effective cushion (lower stiffness, lower coefficient of restitution) shall be analyzed for pile capacity calculations. If accurate values are not known, parameter changes of 25% from nominal may be tried. Wood chips as a hammer cushion are totally unpredictable and therefore shall not be allowed. This is particularly true when the wave equation is used for construction control.

In recent years, uncushioned hammers have been used with increasing frequency. For the wave equation analysis without cushion spring, the stiffness of the spring between hammer and helmet is derived from either ram or impact block (diesels). This stiffness is very high, much higher than the stiffnesses of most other components within the system, may lead to inaccurate stress predictions. Analyses with different numbers of pile segments will show the sensitivity of the numerical solution.

4.5.5 - Soil parameter Selection

The greatest errors in ultimate capacity predictions are usually observed when the soil resistance has been improperly considered. A very common error is the confusion of design loads with the wave equation's ultimate capacity. Note that the wave equation capacity must be divided by a factor of safety to produce the allowable design load.

Since the soil is disturbed at the end of driving, it often has a lower capacity at that time. For this reason, a restrike test shall be conducted to assess the ultimate pile capacity after time dependent soil strength changes have occurred. However, restrike testing is not always easy. The hammer is often not warmed up and only slowly starts to deliver the expected energy as the same time the bearing capacity of the soil deteriorates. Depending on the sensitivity of the soil, the driving resistance may be taken from the first 75 mm of pile penetration even though this may be conservative for some sensitive soils. For construction control, it is more reasonable to develop a site specific setup factor in a preconstruction test program. As long as the hammer is powerful enough to move the pile during restrike and mobilize the soil resistance, restrike tests with dynamic measurements are an excellent tool to calculate setup factors. For the production pile installation, the required end of driving capacity is the required ultimate capacity divided by the setup factor. Using the wave equation analysis and the reduced end of driving capacity, the required end of driving blow count is then calculated.

Although the proper consideration of static resistance at the time of driving or restriking is of major importance for accurate results, dynamic soil resistance sometimes play an important role. Damping factors have been observed to vary with waiting times after driving. Thus, damping factors may have to be chosen for analyzing modeling in restrike situations. Studies on this subject are still continuing, damping factors are not a constant for a given soil type. For soft soils, they may be much higher than recommended and on hard rock they may be much lower. Choosing a low damping factor may produce non-conservative capacity predictions.

Shaft quakes are usually satisfactory as recommended at 2.5 mm. However, larger toe quakes than the typically recommended pile diameter divided by 120 may have to be chosen, particularly when the soil is rather sensitive to dynamic effects. Only dynamic measurements can appear a more accurate magnitude of soil quakes. However, conservative assumptions sometimes have to be made to protect against unforeseen problems. Fortunately, toe quakes have a relatively insignificant effect on the wave equation of piles having most of their resistance acting along the shaft. However, for end bearing piles particularly displacement piles, large toe quakes often develop during driving in saturated soils causing the toe resistance to build up only very slowly during the hammer blow. Thus, at the first instant of stress wave arrival at the pile toe, little resistance exists and tension stresses can develop. In the case of concrete piles, the tension stresses can produce pile damage. At the same time, large toe quakes dissipate an unusually large amount of energy and therefore cause high blow counts. Thus, more cushioning or lower hammer strokes may not be a possible alternative for stress reductions. In extreme cases, hammers with heavier rams and lower strokes had to be chosen to reduce the negative effects of large toe quakes.

Stress predictions particularly tension stresses are also sensitive to the input of the resistance distribution and to the percentage of toe resistance. If the soil resistance distribution is based on a static analysis, then chances are the shaft resistance is set too high because of the loss of shaft resistance during driving. It is therefore recommended the drivability analyses be performed with shaft resistances reduced by estimated setup factors which will adjust the statically calculated capacity to the conditions occurring during driving.

Residual stress wave equation analyses are superior to normal analyses in basic concept and probably in results. Unfortunately, not enough correlation work has been performed to empirically determine dynamic soil constants (quakes and dampings) that shall be used with residual stress analyses. Another reason for its slow acceptance is the slower analysis performance. However, for long slender piles with significant shaft resistance components, residual stress analyses shall be performed to assess potentially damaging stress conditions and the possibility of ultimate capacities which can be much higher than indicated by the standard wave equation analysis. Note that residual stress analyses may not be meaningful to represent early restrike situations where energies increase from blow to blow while, in sensitive soils, capacities successively decrease. The residual stress analysis assumes that hammer energy and pile capacity are constant under several hammer blows.

4.5.6 - Comparison with dynamic measurements

The first impression is that wave equation predicted stresses and bearing capacity agree quite well with results from field dynamic measurements. However, there are additional observations and measurements shall be compared, such as stroke, bounce chamber pressure, and transferred energy. Transferred energy values are often slightly lower than calculated and adjustment of hammer efficiency may improve energy but produce problems with driving stress and bearing capacity. Therefore, instead of adjusting hammer efficiency, the coefficients of restitution may have to be lowered. Sometimes, matching of measured values can be very complicating and difficult. Matching stresses and transferred energies within 10% of the observed or measured quantities may be accurate enough. The wave equation maximum stresses in the final summary table can be along to the length of the pile and may not occur at that same location as the field measured maxima occur. When comparing with GRLWEAP and field measurement results, it is important to check the driving stresses in the extreme tables for the pile segment that corresponds to the measurement location.

In summary, the following procedure is suggested for matching wave equation predictions with field measurements:

- All adjustments are done until the quantities to be matched agree within 10%. It is to be realized that CAPWAP and GRLWEAP work with different models and input quantities and therefore cannot agree perfectly.

- Perform wave equation modeling as accurately as possible for the system which measurements are taken. Use observed stroke, CAPWAP bearing capacity and associated soil parameters and cushion properties as per standard recommended values.

- For matching of transferred energy, vary hammer efficiency by increasing it to maximum 95% and decreasing it to no less than 50% of the standard recommended hammer efficiency for that hammer type. If efficiency changes are insufficient to produce agreement between wave equation calculation and field measurement results to within 10%, adjust cushion coefficients. The cushion coefficients shall not be increased to values above 98% nor decreased to values less than 50% of the standard recommended coefficient for that cushion material.

- For matching the measured force, adjust cushion stiffness (pile cushion if present otherwise hammer cushion). This process may require readjusting hammer efficiency and coefficient for energy match as per above steps. Additional correlations through above steps shall be made until transferred energy and force are within 10%.

- Compare blow counts. Change the shaft and toe damping and the toe quake simultaneously and proportionately to achieve agreement between measured and computed blow counts.

CHAPTER 5 - DYNAMIC PILE TESTING AND ANALYSIS

5.1 - Introduction

Dynamic test method uses measurements of strain and acceleration taken near the pile head as a pile is driven or restrike with a pile driving hammer. These dynamic measurements can be used to evaluate the performance of the pile driving system, calculate pile installation stresses, determine pile integrity, and estimate static pile capacity.

Dynamic test results can be further evaluated using signal matching techniques to determine the soil resistance distribution on the pile, as well as representative dynamic soil properties for use in wave equation analyses. This chapter provides a brief discussion of the equipment and methods of analysis associated with dynamic measurements.

5.2 - Background

The development of the dynamic pile testing techniques is known as the Case Method which is started with a Master thesis project at Case Institute of Technology. This work has been done by Eiber (1958) at the suggestion and under the direction of Professor H. R. Nara. In this first project, a laboratory study has been performed that a rod was driven into dry sand. The Ohio Department of Transportation (ODOT) and the Federal Highway Administration subsequently funded a project with HPR funds at Case Institute of Technology beginning in 1964. This project is directed by Professors R. H. Scanlan and G. G. Goble. At the end of the first two year phase, Professor Scanlan moved to Princeton University, the research work at Case Institute of Technology under the direction of Professor Goble continued to be funded by ODOT and FHWA, as well as several other public and private organizations until 1976.

Four principal directions were researched during the 12 year period that the funded research project was activated. There was a continuous effort to develop improved transducers for the measurement of force and acceleration during pile driving. Field equipment for recording and data processing was also continually improved. Model piles driven and statically tested by ODOT, and later other DOT’s, were also tested dynamically to obtain capacity correlations. Finally, analysis method improvements were developed, including both field solutions (Case Method) and a strict numerical modeling technique (CAPWAP).

ODOT began to apply the results of this research to their construction projects in about 1968. Commercial use of the methods began in 1972 when the Pile Driving Analyzer (PDA) and CAPWAP became practical for use in routine field testing by a trained engineer. There have been continual improvements in the hardware since 1972, making the equipment more reliable and easier to use. Further performance of dynamic testing methods resulted from FHWA Demonstration Project 66, the method benefits were demonstrated on real projects throughout the US. Subsequently, other dynamic testing and analysis systems have been developed such as the FPDS equipment and its associated signal matching technique. However, based on the current state of practice in the United States, this technique will focus on the Pile Driving Analyzer and CAPWAP because of specific advantages relating to the comparatively extensive correlation database with static tests.

5.3 - Applications for dynamic testing methods

Cheney and Chassie (1993) note that dynamic testing costs much less and requires less time than static pile load testing. They also note that important information can be obtained regarding to the behavior of the pile driving hammer and pile-soil systems when it is not available from static pile load test. Consequently, dynamic testing has many applications. Some of these applications are discussed below.

5.3.1 - Static pile capacity

- Evaluation of static pile capacity at the time of testing. The soil setup or relaxation potential can be assessed by restriking several piles and comparing restrike capacities with the end of initial driving capacities.

- Assessments of static pile capacity versus pile penetration depth can be obtained by testing from the start to the end of driving. This can be helpful in profiling the depth to the bearing strata and the required pile lengths.

- CAPWAP analysis can provide refined estimation of static capacity, assessment of soil resistance distribution, and soil quake and damping parameters for wave equation input.

5.3.2 - Hammer and driving system performance

- Calculation of energy transferred to the pile for comparison with the manufacturer's rated energy and wave equation predictions which indicate hammer and drive system performance. Energy transfer can also be used to determine effects of changes in hammer cushion or pile cushion materials on pile driving resistance.

- Determination of drive system performance under different operation, such as pressures, strokes or batters, or changes in hammer maintenance by comparative testing of hammers or of a single hammer over an extended period of use.

- Identification of hammer performance problems, such as pre-ignition problems with diesel hammers or pre-admission in air / stream hammers.

- Determination of whether soil behavior or hammer performance is responsible for changes in observed driving resistances.

5.3.3 - Driving stresses and pile integrity

- Calculation of compression and tension driving stresses. In case with driving stress problems, this information can be helpful when evaluating adjustments to pile installation procedures. Calculated stresses can also be compared to specified driving stress limits.

- Determination of the extent and location of pile structural damage. Thus, costly extraction may not be necessary to confirm or quantify damage suspected from driving records.

- CAPWAP analysis for stress distribution along the pile.

5.4 - Dynamic testing equipment

A typical dynamic testing system consists of a minimum of two strain transducers and two accelerometers bolted on the opposite sides of pile to monitor strain and acceleration. It is in order to account for non-uniform hammer impacts and pile bending. The reusable strain transducers and accelerometers and generally attached two to three diameters below the pile head. Almost any driven pile type (concrete, steel pile, H, Monotube, timber, etc.) can be tested with pile preparation for each pile types slightly varying.

Figure 5-1 and Figure 5-2 illustrate that the typical pile preparation procedure required for dynamic testing. In figure 5-1, a prestressed concrete pile is being prepared for gage attachment by drilling and then installing concrete anchors. In figure 5-2, the concrete pile to be tested during driving has been positioned in the leads for driving. Piles to be tested during restrike can be instrumented at any convenient location and the climbing of the leads in usually not necessary. Pile preparation and gage attachment typically requires 10 to 20 minutes per pile tested. After the gages are attached, the driving or restrike process continues following usual procedures. Most restrike tests are only 20 blows or less.

Figure 5-1 – Pile preparation for dynamic testing

Resource: Pile Dynamics, Inc

Figure 5-2 – Gages bolted on pile

Resource: Pile Dynamics, Inc

Figure 5-3 – Strain transducer and accelerometer bolted on the concrete pile

Resources: Garlan Likins, P.E. Advances in Dynamic Foundation Testing Technology

Figure 5-4 – Pile driving analyzer (courtesy of Pile Dynamics, Inc)

Resource: Pile Dynamics, Inc

A close up view of strain transducer and an accelerometer bolted to a steel pipe pile is shown in Figure 5-3. The individual cables from each gage are combined into a single main cable which in turn relays the signals from each hammer blow to the data acquisition system on the ground. The data acquisition system, such as the Pile Driving Analyzer shown in Figure 5-4, conditions and converts the strain and acceleration signals to force and velocity records versus time. The force is computed from the measured strain, ε, times the product of the pile elastic modulus, E, and cross sectional area, A, then,

$$F(t) = EA\varepsilon(t) \tag{5-1}$$

where ε = Measured strain

A = Cross sectional area

E = Pile elastic modulus

The velocity is obtained by integrating the measured acceleration, record, a, then,

$$V(t) = \int a(t)dt \tag{5-2}$$

where a = Acceleration

Older dynamic testing systems required multiple components for processing, recording, and display of dynamic test signals. In newer dynamic testing systems, these components have been combined into one PC computer based system.

During driving, the Pile Driving Analyzer performs integrations and all other required computations to analyze the dynamic records for transferred energy, driving stresses, structural integrity and pile capacity. Numerical results for each blow for up to nine dynamic quantities are electronically stored in a file which can be later used to produce graphical and numeric summary outputs. In this system, force and velocity records are also viewed on a graphic LCD computer screen during pile driving to evaluate data quality, soil resistance distribution and pile integrity. Complete force and velocity versus time records from each gage are also digitally stored for later reprocessing and data analysis by CAPWAP.

Data quality is automatically evaluated by the Pile Driving Analyzer and if any problem is detected, then a warning is given to the test engineer. Other precautionary advice is also displayed to assist the engineer in collecting data. The capabilities discussed in the remainder of this chapter are those included in these newer systems.

Additional information on the equipment requirements for dynamic testing are detailed in ASTM D-4945, Standard Test Method for High Strain Dynamic Testing of Piles and in AASHTO T-298-33, Standard Method of Test for High Strain Dynamic Testing of Piles.

5.5 - Basic wave mechanics

This section is summarizing wave mechanics principles applicable to pile driving. Through this overview, an understanding of how dynamic testing functions and how test results be qualitatively interpreted can be obtained.

When a uniform elastic rod of cross sectional area, A, elastic modulus, E, and wave speed, C, is struck by a mass, then a force, F, is generated at the impact surface of the rod. This force compresses the adjacent part of the rod. Since the adjacent material is compressed, it also knows the acceleration or a particle velocity, V. As long as there are no resistance effects on the uniform rod, the force in the rod will be equal to the particle velocity times the rod impedance, Z.

$$Z = \frac{EA}{C} \tag{5-3}$$

where C = Wave speed

A = Cross sectional area

E = Pile elastic modulus

Figure 5-5(a) illustrates a uniform rod of length, L, with no resistance effects, that is struck at one end by a mass. Force and velocity (particle velocity) waves will be created in the rod, as shown in Figure 5-5(b). These waves will then travel down the rod at the material wave speed, C. At time L/C, the waves will arrive at the end of the rod, as shown in Figure 5-5(c) and Figure 5-5(d). Since there are no resistance effects acting on the rod, a free end condition exists, and a tensile wave reflection occurs, which doubles the pile velocity at the free end and the net force becomes zero. The wave then travels up the rod with force of the same magnitude as the initial input, except in tension, and the velocity of the same magnitude and same sign.

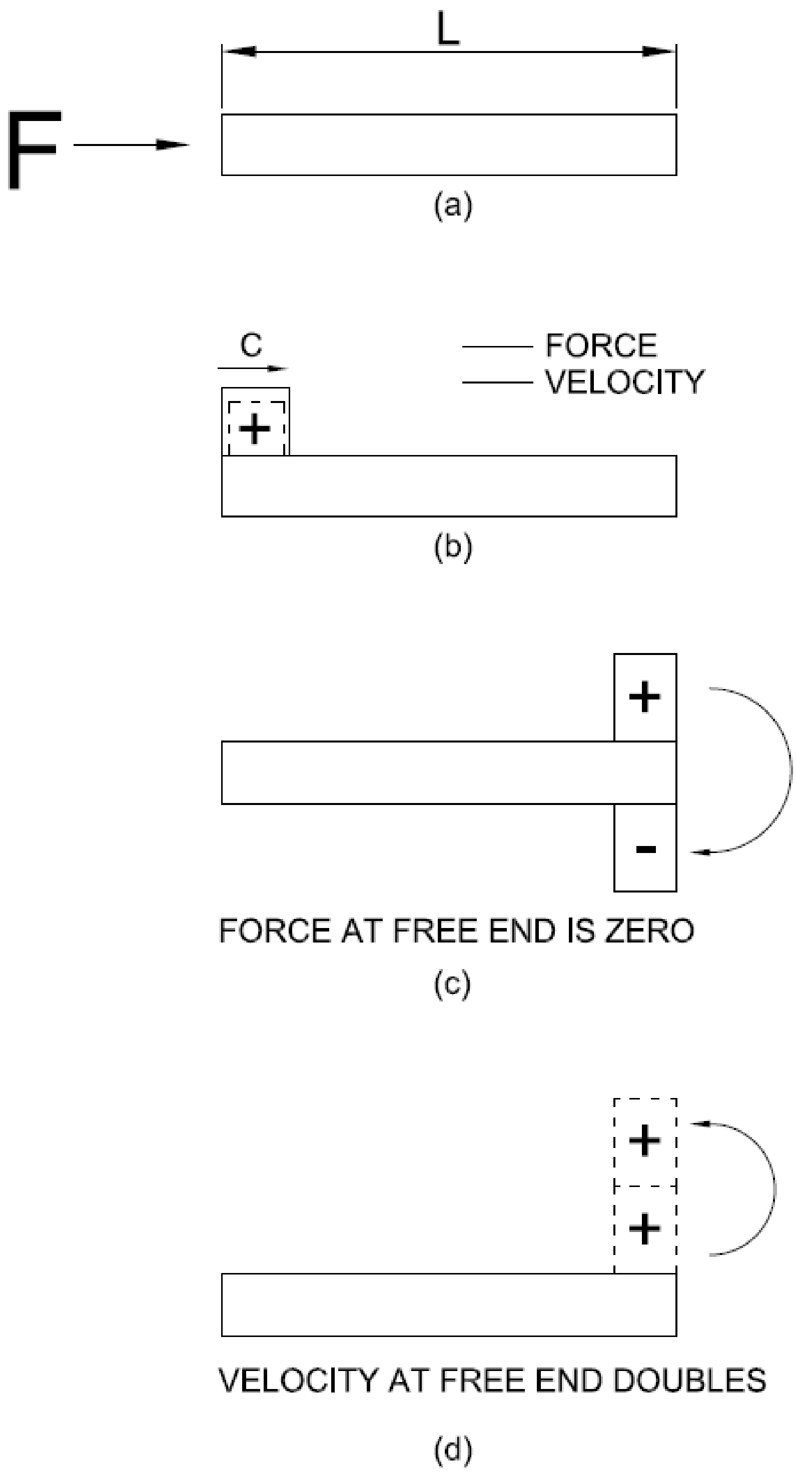


Figure 5-5 – Free end wave mechanics

Resource: CAPWAP manual

Consider now that the rod is a pile with no resistance effects, and that force and velocity measurements are made near the pile head. Force and velocity measurements versus time for this “free end” condition are presented in Figure 5-6. The toe response in the records occurs at time 2L/C. This is the time required for the waves to travel to the pile toe and back to the measurement location, divided by the wave speed. Since there are no resistance effects acting on the pile shaft, the force and velocity records are equal until the reflection from the free end condition arrives at the velocity wave doubles in magnitude. Note the repetitive pattern in the records at 2L/C intervals generated as the waves continue to travel down and up the pile. This illustration is typical of an easy driving situation where the pile “runs” under the hammer blow.

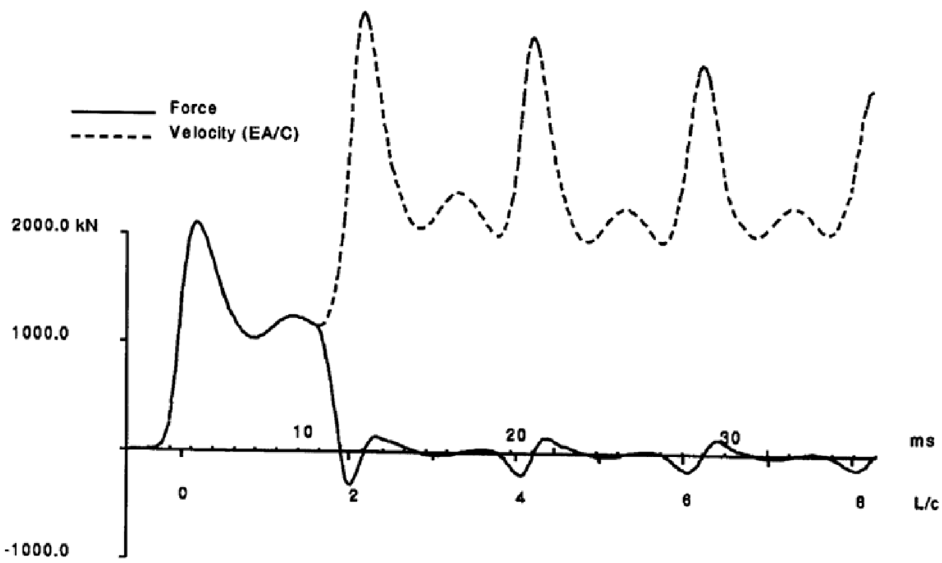


Figure 5-6 – Force and Velocity measurements versus Time for Free End Condition

Resource: CAPWAP manual

Figure 5-7(a) illustrates a uniform rod of length, L, that is struck by a mass. Again there are no resistance effects along the rod length, but the pile end is fixed, i.e., it is prevented by some mechanism from moving in such a manner that the particle velocity must be zero at that point. The mass impact will impart force and velocity waves in the rod as shown in figure 5-7(b). These waves will again travel down the rod at the material wave speed, C. At time L/C, the waves will arrive at the end of the rod as shown in Figure 5-7(c) and Figure 5-7(d). There the fixed end condition will cause a compression wave reflection and therefore the force at the fixed end doubles in magnitude and the pile velocity becomes zero. A compression wave then travels up the rod.

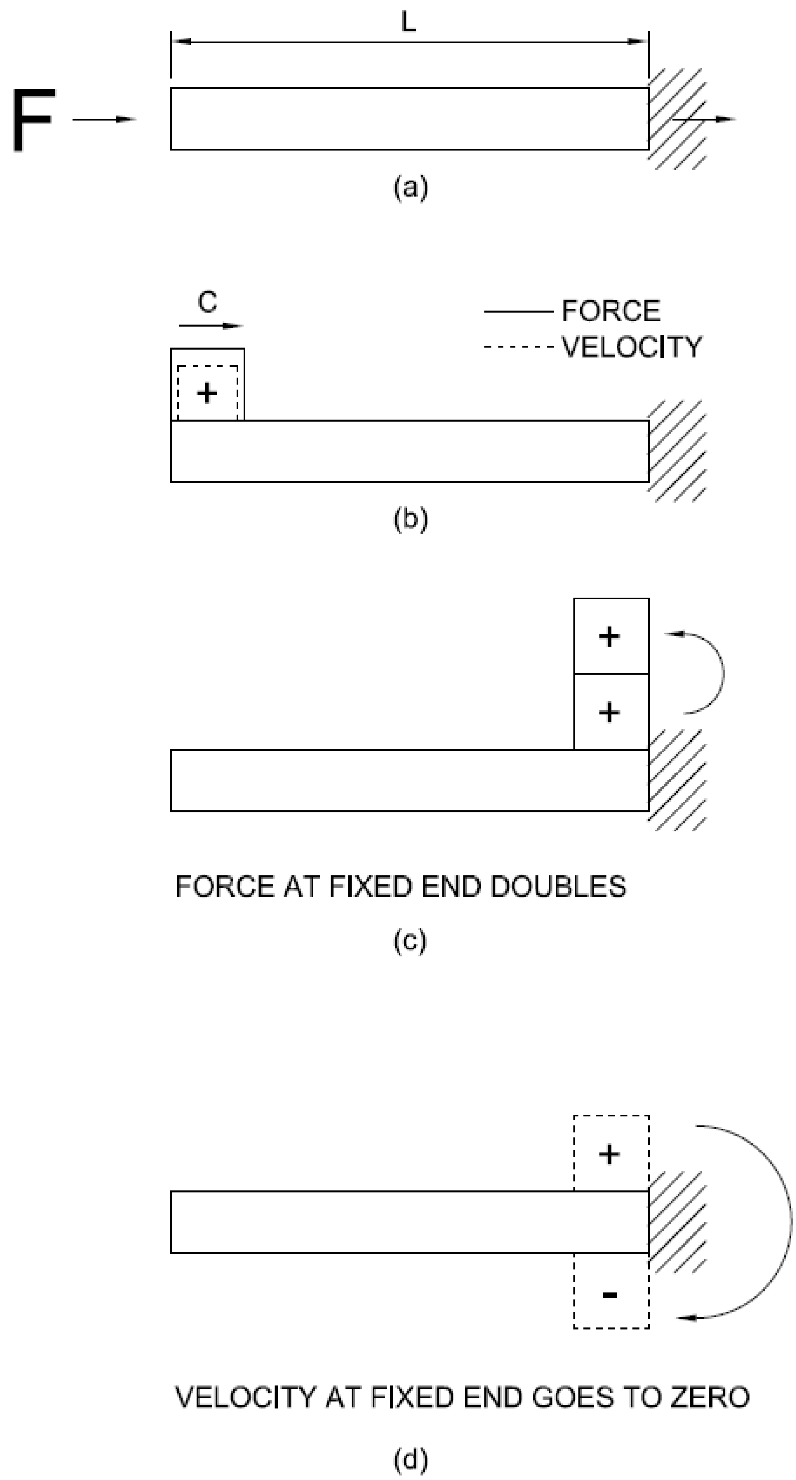


Figure 5-7 – Fixed end wave mechanics

Resources: CAPWAP manual

Consider now that the rod is a pile with a fixed end condition and that force and velocity measurements are again made near the pile head. The force and velocity measurements versus time for this condition are presented in Figure 5-8. Since there are no resistance effects acting on the pile shaft, the force and velocity records are equal until the reflection from the fixed end condition arrives at the measurement location. At time 2L/C, the force wave increases in magnitude and the velocity wave goes to zero. This illustration is typical of a hard driving situation where the pile is driven to rock.

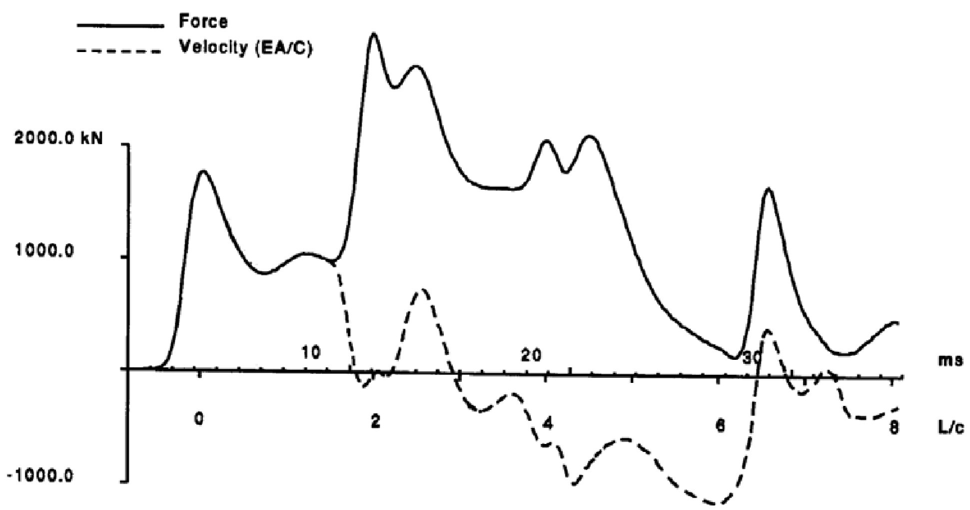


Figure 5-8 – Force and Velocity measurements versus time for Fixed End Condition

Resource: CAPWAP manual

As discussed above, the force and velocity records versus time are equal or proportional at impact and remain proportional thereafter until affected by soil resistance or cross sectional changes. Reflections from either effect will arrive at the measurement location at time 2X/C where X is the distance to the soil resistance or cross section change. Both soil resistance effects and cross sectional increases will cause an increase in the force record and a proportional decrease in the velocity record. Conversely, cross sectional reductions, such as those caused by pile damage, will cause a decrease in the force record and an increase in the velocity record.

The concept of soil resistance effects on force and velocity records can be further understood by reviewing the theoretical soil resistance example presented in Figure 5-9. In this case, the soil resistance on a pile consists only of a small resistance located at a depth, A, below the measurement location, and a larger soil resistance at depth B. No other resistance effects act on the pile, so a free end condition is present at the pile toe. The force and velocity records versus time for this example will be proportional until time 2A/C, when the reflection from the small soil resistance effect arrives at the measurement location. This soil resistance reflection will then cause a small increase in the force record and a small decrease in the velocity record.

No additional soil resistance effects act on the pile between times 2A/C and 2B/C. Therefore, the force and velocity records will remain parallel over this time interval with no additional separation. At time 2B/C, the reflection from the large soil resistance reflection will then cause a large increase in the force record and a large decrease in the velocity record. No additional soil resistance effects act on the pile between time 2B/C and 2L/C. Therefore, the force and velocity records will again remain parallel over this time interval with no additional separation between the records.

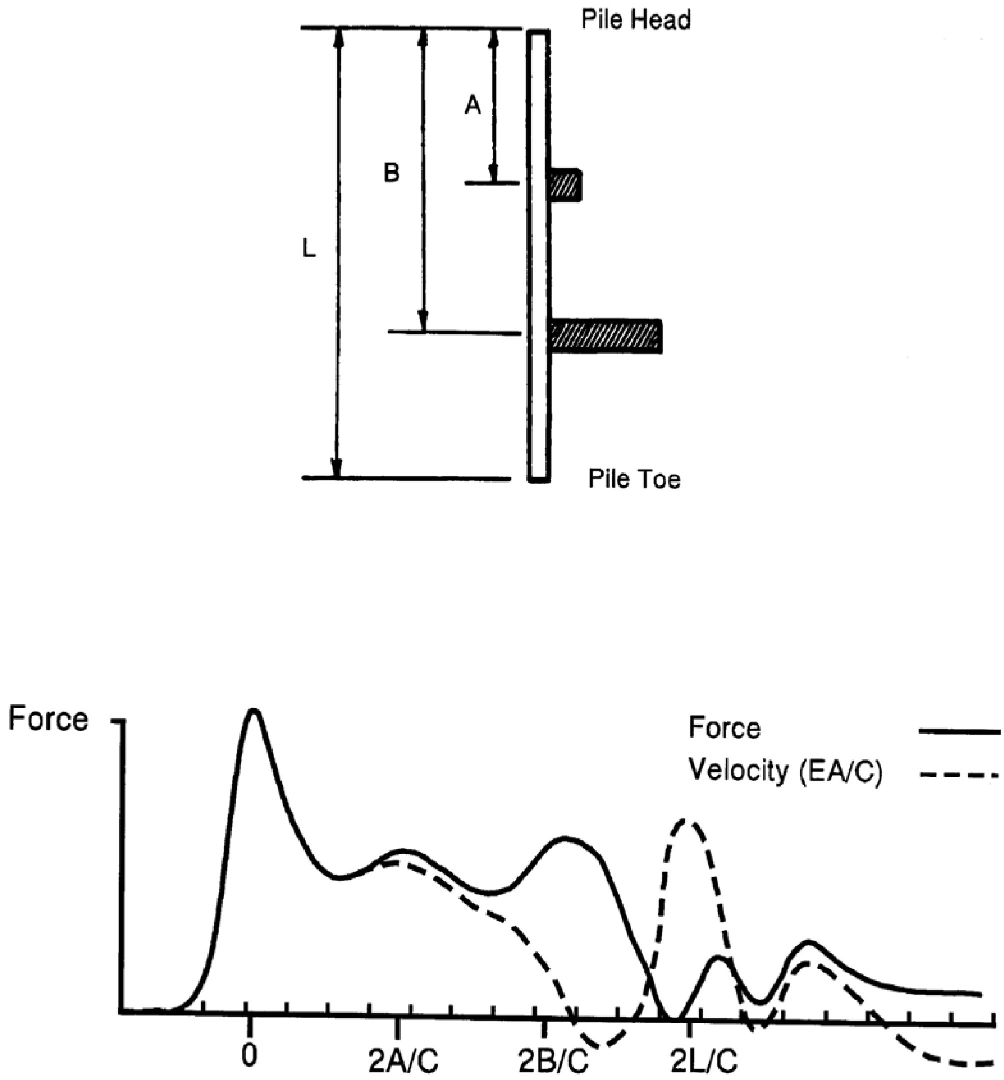


Figure 5-9 – Soil resistance effects on force and velocity records

Resource: CAPWAP manual

At time 2L/C, the reflection from the pile toe will arrive at the measurement location. Since no resistance is present at the pile toe, a free end condition exists and a tensile wave will be reflected. Hence, an increase in the velocity record and a decrease in the force record will occur.

These base interpretation concepts of force and velocity records versus time can be used to qualitatively evaluate the soil resistance effects on a pile. In Figure 5-10(a), minimal separation occurs between the force and velocity records between time 0, or the time of impact, and time 2L/C. In addition, a large increase in the velocity record and corresponding decrease in the force record occurs at time 2L/C. Hence, this record indicates minimal shaft and toe resistance on the pile.

In Figure 5-10(b), minimal separation again occurs between the force and velocity records between time 0 and time 2L/C. however in this example, a large increase in the force record and corresponding decrease in the velocity record occurs at time 2L/C. Therefore, this force and velocity record indicates minimal shaft and a large toe resistance on the pile.

In Figure 5-10(c), a large separation between the force and velocity record between time 0 and time 2L/C. This force and velocity record indicates a large shaft resistance on the pile.

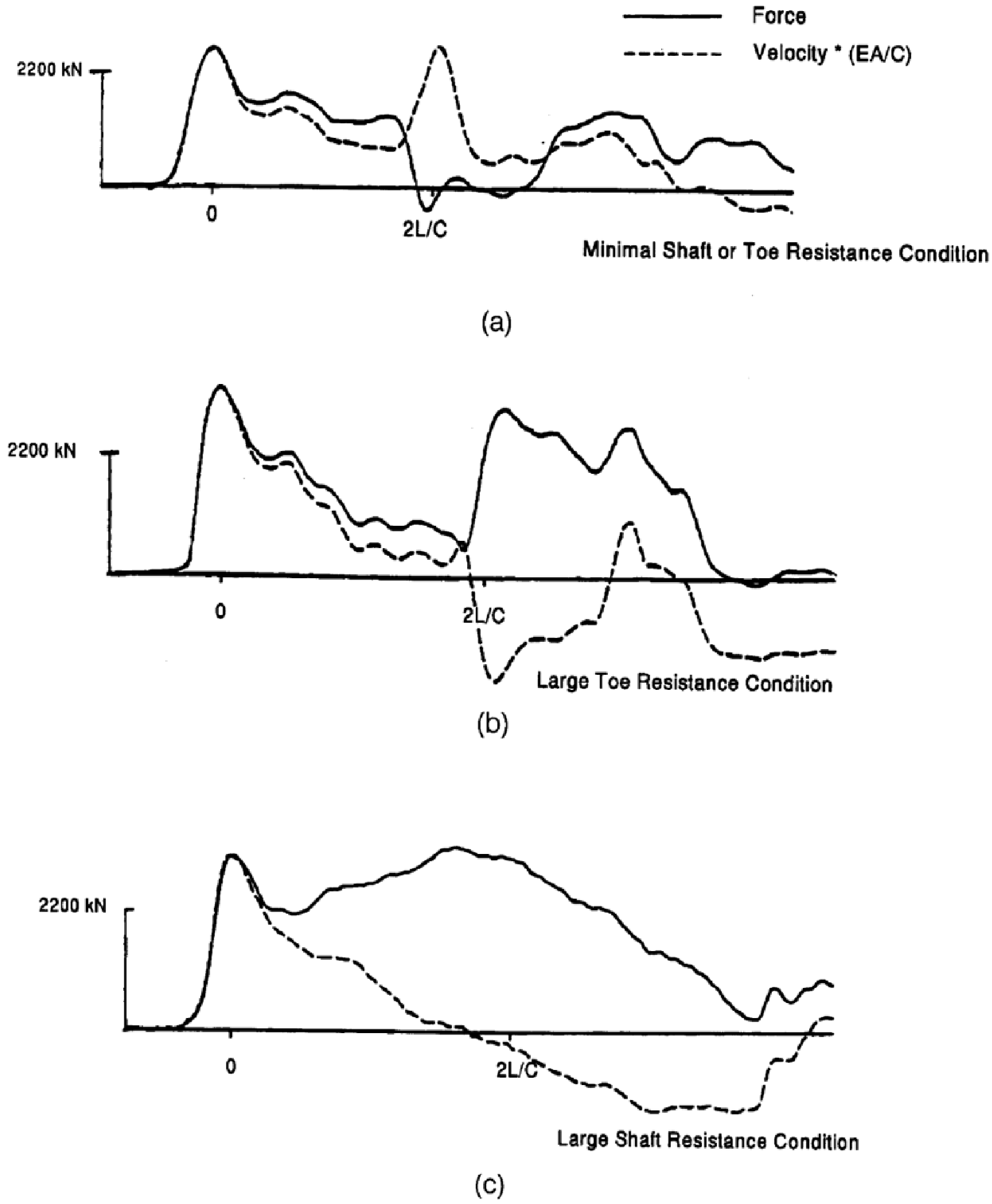


Figure 5-10 – Typical force and velocity records for various soil resistance condition

Resource: CAPWAP manual

5.6 - Dynamic testing methodology

As introduced in Section 5-1, two methods have developed for analyzing dynamic measurement data, the Case Method and CAPWAP. In the field, the pile Driving Analyzer uses the Case Method equations for estimates of static pile capacity, calculation of driving stresses and pile integrity, as well as computation of transferred hammer energy. The CAPWAP analysis method is a more rigorous numerical analysis procedure that uses dynamic records of force and velocity along with wave equation type pile and soil modeling to calculate static pile capacity, the relative soil resistance distribution, and dynamic soil properties of quake and damping. Static pile capacity evaluation from these two methods will be described in greater detail in subsequent sections. For additional details of the dynamic analysis procedures, references are provided at the end of this chapter.

5.6.1 - Case method capacity

Research conducted at Case Western Reserve University in Cleveland, Ohio, resulted in a method which uses electronic measurements taken during pile driving to predict static pile capacity. Assuming the pile is linearly elastic and has constant cross section, the total static and dynamic resistance on a pile during driving, RTL, can be expressed using the following equation, which was derived from a closed form solution to the one dimensional wave propagation theory:

$$\mathrm{RTL} = \frac{1}{2}[\mathrm{F}(\mathrm{t}_1) + \mathrm{F}(\mathrm{t}_2)] + \frac{1}{2}[\mathrm{V}(\mathrm{t}_1) - \mathrm{V}(\mathrm{t}_2)]\frac{\mathrm{EA}}{\mathrm{C}} \quad (5\text{-}4)$$

where F = Force measured at gage location

V = Velocity measured at gage location

t_1 = Time of initial impact

t_2 = Time of reflection of initial impact from pile toe (t_1 + 2L/C)

E = Pile modulus of elasticity

C = Wave speed of pile material

A = Pile area at gage location

L = Pile length below gage location

To obtain the static pile capacity, the dynamic resistance (damping) must be subtracted from the above equation. Goble et al. (1975) found that the dynamic resistance component could be approximated as a linear function of a damping factor times the pile toe velocity, and that the pile toe velocity could be estimated from dynamic measurements at the pile head. This led to the standard Case Method capacity equation, RSP, expressed below:

$$\mathrm{RSP} = \mathrm{RTL} - \mathrm{J}\left[\mathrm{V}(\mathrm{t}_1)\frac{\mathrm{EA}}{\mathrm{C}} + \mathrm{F}(\mathrm{t}_1) - \mathrm{RTL}\right] \quad (5\text{-}5)$$

where J = Dimensionless damping factor based on soil type near the pile toe

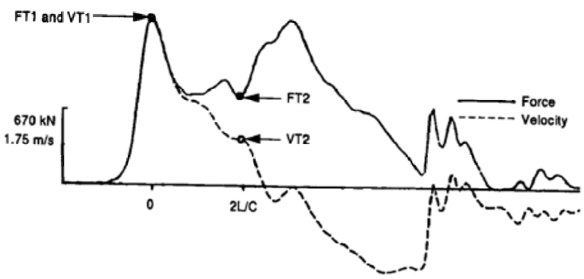
Typical damping factors versus soil type at the pile toe were determined by finding the range in the Case damping factor, J, for a soil type that provided a correlation of the RSP static capacity within 20% of the static load test failure load, determined using the Davisson (1972) offset limit method. The original range in Case damping factor versus soil type from this correlation study, Goble et al. (1975), as well as typical ranges in Case damping factor for the RSP equation based on subsequent experience, Pile Dynamics, Inc. (1996), are presented in Table 5-1. While use of these values with the RSP equation may provide good initial capacity estimates, site specific damping correlations should be developed based upon static load test results or CAPWAP analysis. It should also be noted that Case damping is a non-dimensional damping factor and is not the same as the Smith damping discussed in Chapter 4 for wave equation analysis.

Soil type at pile toe	**Original Case damping correlation Range Goble et al. (1975)**	**Updated Case damping ranges Pile Dynamics (1996)**
Clean Sand	0.05 – 0.20	0.10 – 0.15
Silty sand, sand silt	0.15 – 0.30	0.15 – 0.25
Silt	0.20 – 0.45	0.25 – 0.40
Silty clay, clayey silt	0.40 – 0.70	0.40 – 0.70
Clay	0.60 – 1.10	> 0.70

Table 5-1 – Summary of Case damping factors for RSP equation

The RSP or standard Case Method equation is best used to evaluate the capacity of low displacement piles, and piles with large shaft resistances. For piles with large toe resistances and for displacement piles driven in soils with large toe quakes, the toe resistance is often delayed in time. This condition can be identified from the force and velocity records. In these instances, the standard Case Method equation may indicate a relatively low pile capacity and the maximum Case Method equation, RMX, should be used. The maximum Case Method equation searches for the t, time in the force and velocity records which results in the maximum capacity. An example of this technique is presented in Figure 5-11. When using the maximum Case Method equation, experience has shown that the Case damping factor should be at least 0.4, and on the order of 0.2 higher than that used for the standard Case Method capacity equation, RSP.

STANDARD CASE METHOD, RSP



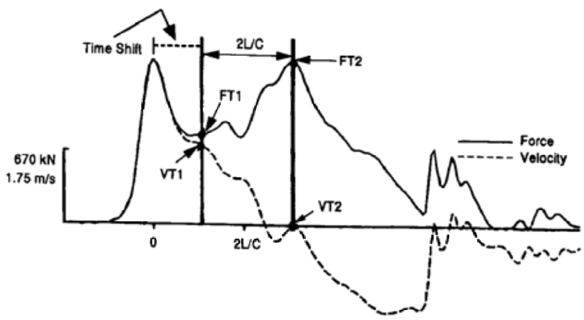
TOTAL RESISTANCE

RTL = 1/2 (FT1 + FT2) + 1/2 (VT1 - VT2) (EA/C)
= 1/2 (1486 + 819) + 1/2 (3.93 -1.07) 381
= 1153 + 545 = 1698 kN.

STATIC RESISTANCE

RSP = RTL - J[VT1 (EA/C) + FT1 - RTL]
= 1698 - 0.4 [3.93 (381) + 1486 - 1698]
= 1698 - 514 = 1184 kN.

MAXIMUM CASE METHOD, RMX



TOTAL RESISTANCE

RTL = 1/2 (FT1 + FT2) + 1/2 (VT1 - VT2) (EA/C)
= 1/2 (819 + 1486) + 1/2 (1.92 - 0.0) 381
= 1153 + 366 = 1519 kN.

STATIC RESISTANCE

RMX = RTL - J[VT1 (EA/C) + FT1 - RTL]
= 1519 - 0.7 [1.92 (381) + 819 - 1519]
= 1519 - 22 = 1497 kN.

Figure 5-11 – Standard, RSP and Maximum, RMX, Case Method Capacity Estimates

Resource: CAPWAP manual

The RMX and RSP Case Method equations are the two most commonly used solutions for field evaluation of pile capacity. Additional automatic Case Method solutions are available that do not require selection of a Case damping factor. These automatic methods, referred to as RAU and RA2, search for the time when the pile toe velocity is zero and hence damping is minimal. The RAU method may be applicable for pile with minimal shaft resistance and the RA2 method may be applicable to piles with toe resistance plus moderate shaft resistance. It is recommended that these automatic methods be used as supplemental indicators of the pile capacity where appropriate with the more traditional standard or maximum Case Method equations primarily used to evaluate pile capacity.

5.6.2 - Energy transfer

The energy transferred to the pile head can be computed from the strain and acceleration measurements. As described in section, the acceleration signal is integrated to obtain velocity and the strain measurement is converted to force. Transferred energy is equal to the work done which can be computed from the integral of the force and velocity records over time as given below:

$$E_p(t) = \int_0^t F(t)V(t)dt \tag{5-6}$$

where E_p = The energy at the gage location expressed as a function of time

F = The force at the gage location expressed as a function of time

V = The velocity at the gage location expressed as a function of time

This procedure is illustrated in Figure 5-12. The maximum energy transferred to the pile head corresponds to the maximum value of $E_p(t)$ and can be used to evaluate the performance of the hammer and driving system.

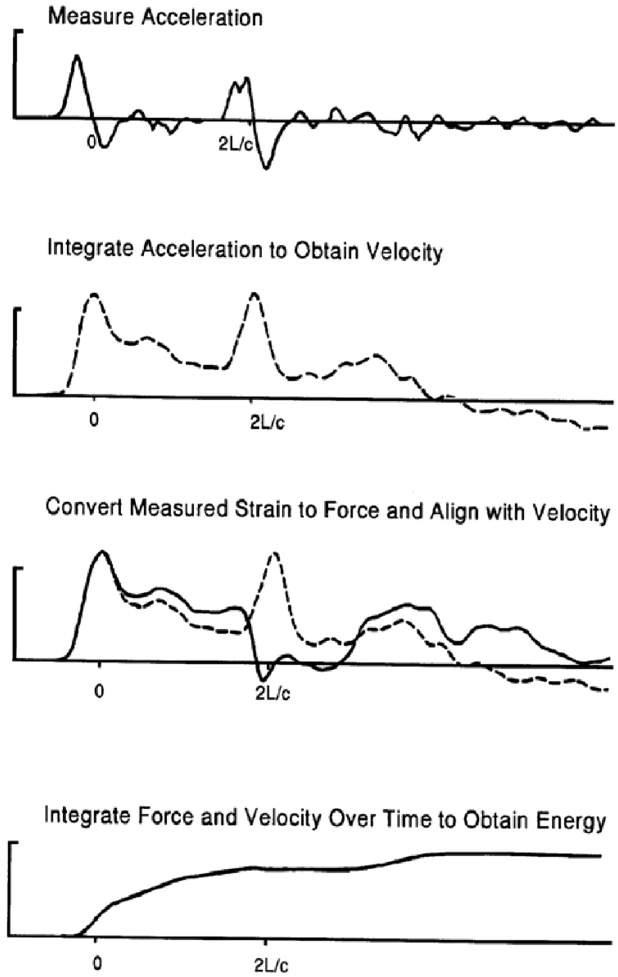


Figure 5-12 – Energy transfer computation

Resource: CAPWAP manual

5.6.3 - Driving stresses and integrity

The Pile Driving Analyzer calculates the compression stress at the gage location using the measured strain and pile modulus of elasticity. However, the maximum compression stress in the pile may be greater than the compression stress calculated at the gage location, such as in the case of a pile driven through soft soils to rock. In these cases CAPWAP or wave equation analysis may be used to evaluate the maximum compression stress in the pile. Computed tension stresses are based upon the superposition of the upward and downward travelling force waves calculated by the Pile Driving Analyzer.

The basic concepts of wave mechanics were presented in Section 5.5. Convergence between the force and velocity records prior to the toe response at time 2L/C indicates an impedance (EA/C) reduction in the pile. For uniform cross section piles an impedance reduction is therefore pile damage. The degree of convergence between the force and velocity records is termed BTA, which can be used to evaluation pile damage following the guidelines presented in Rausche and Goble, (1979). These guidelines are provided in table. Pile with BTA values below 80% correspond to damaged or broken piles.

BTA	**Severity of damage**
1.0	Undamaged
0.8 – 1.0	Slightly damaged
0.6 – 0.8	Damaged
< 0.6	Broken

Table 5-2 – Pile damage guidelines (Rausche and Goble, 1979)

5.6.4 - The CAPWAP method (Case Pile Wave Analysis Program)

CAPWAP is a computer program for a more severe evaluation of static pile capacity, the relative soil resistance distribution, and soil quake and damping characteristics. A CAPWAP analysis is performed on an individual hammer blow that is usually selected from the end of driving or beginning of restrike. A CAPWAP analysis refines the Case Method dynamic test results at a particular penetration depth or time. CAPWAP uses wave equation type pile and soil models; the Pile Driving Analyzer measured force and velocity records are used as the head boundary condition, replacing the hammer model.

In the CAPWAP method depicted in Figure 5-13, the pile is modeled by a series of continuous pile segments and the soil resistance modeled by elasto-plastic springs (static resistance) and dashpots (dynamic resistance). The force and acceleration data from the Pile Driving Analyzer are used to quantify pile force and pile motion, which are two of the three unknowns. The remaining unknown is the boundary conditions, which are defined by the soil model. First, reasonable estimates of the soil resistance distribution and quake and damping parameters are made. Then, the measured acceleration is used to set the pile model in motion. The program then computes the equilibrium pile head force. Initially, the computed and measured pile head forces will not agree with each other. Adjustments are made to the soil model assumptions and the calculation process repeated.

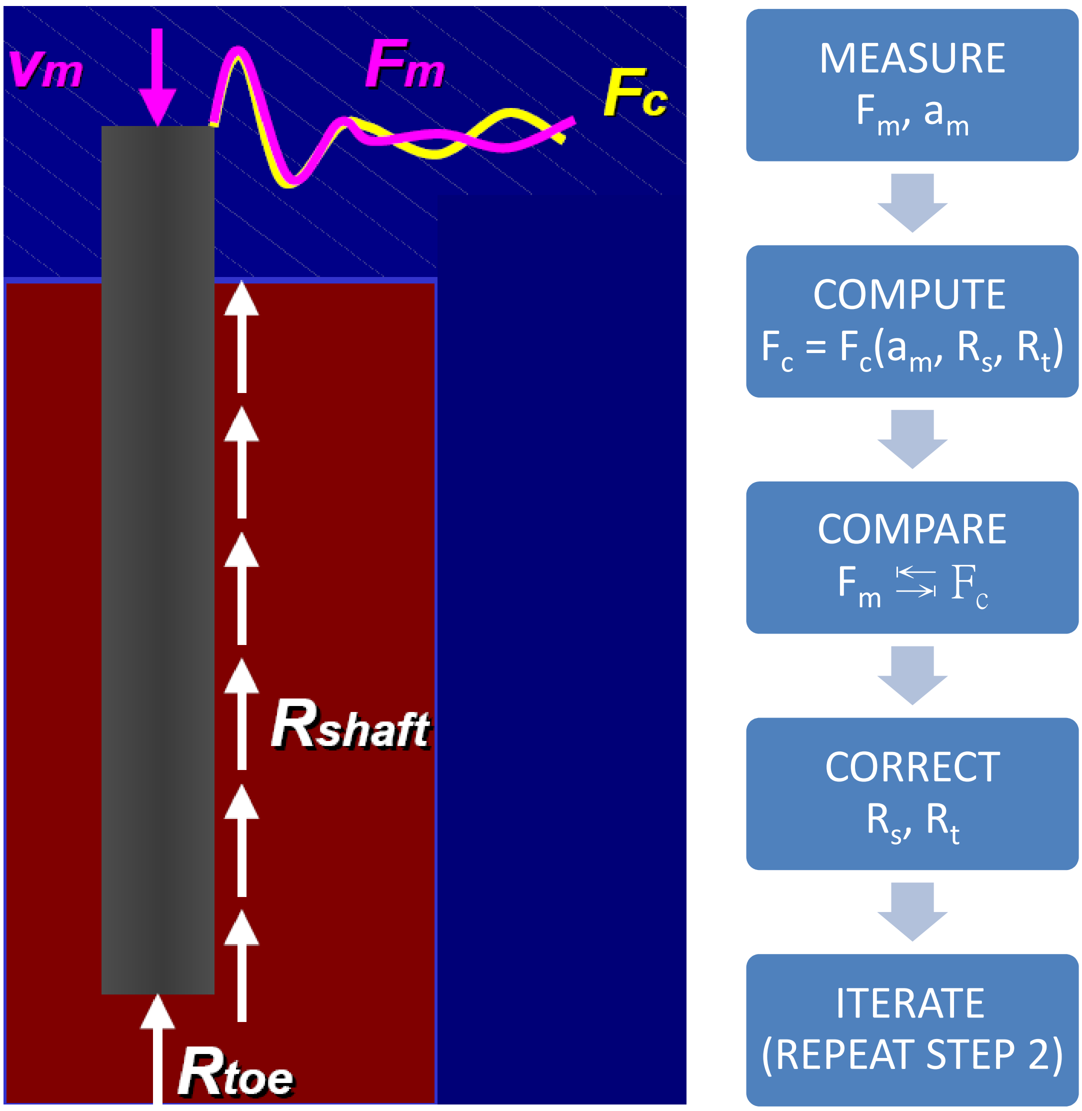


Figure 5-13 – Schematic of CAPWAP analysis method

Resource: Pile Dynamics, Inc

In the CAPWAP matching process, the ability to match the measured and computed waves at various times is controlled by different factors. Figure 5-14 illustrates the factors that most influence match quality in a particular zone. The assumed shaft resistance distribution has the dominant influence on match quality beginning with the rise of the record at time t, before impact and continuing for a time duration of 2L/C thereafter. This is identified as Zone 1.

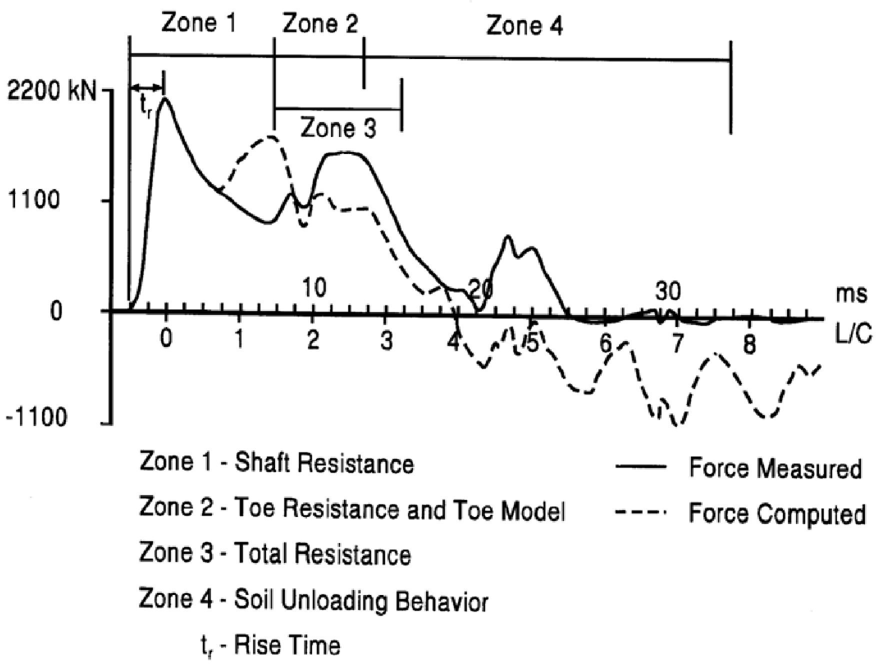


Figure 5-14 – Factors most influencing CAPWAP force wave matching

Resource: CAPWAP manual

In Zone 2, the toe resistance and toe model are (toe damping, toe quake and toe gap) most influence the wave match. Zone 2 begins where Zone

1 ends and continues for the time duration equal to the rise time t, the overall capacity controls the match quality. A good wave match in Zone 3 is essential for accurate capacity assessments. Zone 4 begins at the end of Zone 2 and continues for the duration of about 20ms. The unloading behavior of the soil most influences match quality in this zone.

With each analysis, the program evaluates the match quality by summing the absolute values of the relative differences between the measured and computed waves. The program computes a match quality number for each analysis that is the sum of the individual match quality numbers for each of these four zones. An illustration of the CAPWAP iteration process is presented in Figure 5-15.

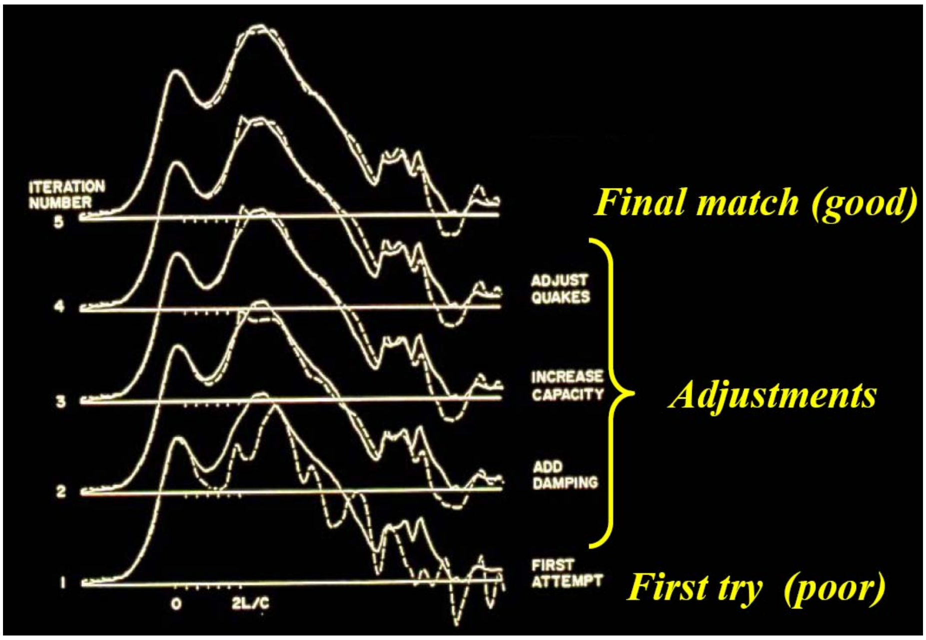


Figure 5-15 – CAPWAP iteration matching process

Resource: Pile Dynamics, Inc

Through this trial and error iteration adjustment process to the soil model as illustrated in figure 5-13, the soil model is refined until no further agreement can be obtained between the measured and computed pile head forces. The resulting soil model is considered the best estimate of the static pile capacity, the soil resistance distribution, and the soil quake and damping characteristics. An example of the final CAPWAP result summary is presented in Figure 5-16. A summary of the stress distribution throughout the pile is also obtained as illustrated in Figure 5-17. Lastly, CAPWAP includes a simulated static load-set graph based on the CAPWAP calculated static resistance parameters and the elastic compression characteristics of the pile.

Galaxy; Pile: EPC80-3 **Test: 25-Jan-2008**
Blow: 12 **CAPWAP?Ver. 2000-1**
Sol Data (Asia) Limited **Simon Wong (30-Jan-2008)**

CAPWAP FINAL RESULTS

Total CAPWAP Capacity: 7931.0; along Shaft 4976.9; at Toe 2954.0 kN

Soil Sgmnt No.	Dist. Below Gages	Depth Below Grade	Ru	Force in Pile	Sum of Ru	Unit Resist. (Depth)	Unit Resist. (Area)	Smith Damping Factor	Quake
	m	m	kN	kN	kN	kN/m	kPa	s/m	mm
				7931.0					
1	3.0	2.5	48.9	7882.1	48.9	24.19	12.83	0.313	9.550
2	5.1	4.6	48.9	7833.2	97.8	24.19	12.83	0.313	9.550
3	7.1	6.6	48.9	7784.3	146.7	24.19	12.83	0.313	9.550
4	9.1	8.6	48.9	7735.4	195.6	24.19	12.83	0.313	9.550
5	11.1	10.6	48.9	7686.5	244.5	24.19	12.83	0.313	9.550
6	13.1	12.6	48.9	7637.6	293.3	24.19	12.83	0.313	9.550
7	15.2	14.7	45.8	7591.8	339.1	22.65	12.02	0.313	9.550
8	17.2	16.7	45.8	7546.1	384.9	22.65	12.02	0.313	9.550
9	19.2	18.7	88.7	7457.4	473.6	43.87	23.28	0.313	9.550
10	21.2	20.7	174.1	7283.3	647.7	86.12	45.69	0.313	9.550
11	23.2	22.7	261.1	7022.2	908.8	129.20	68.54	0.313	9.550
12	25.3	24.8	367.5	6654.6	1276.4	181.83	96.46	0.313	9.550
13	27.3	26.8	459.5	6195.1	1735.9	227.34	120.60	0.313	9.550
14	29.3	28.8	367.5	5827.6	2103.4	181.83	96.46	0.313	9.550
15	31.3	30.8	275.7	5551.8	2379.1	136.42	72.37	0.313	9.550
16	33.4	32.9	55.3	5496.5	2434.4	27.35	14.51	0.313	8.936
17	35.4	34.9	55.3	5441.3	2489.7	27.35	14.51	0.313	7.819
18	37.4	36.9	55.3	5386.0	2545.0	27.35	14.51	0.313	6.702
19	39.4	38.9	55.3	5330.7	2600.3	27.35	14.51	0.313	5.585
20	41.4	40.9	110.9	5219.8	2711.2	54.86	29.10	0.313	4.468
21	43.5	43.0	554.2	4665.6	3265.4	274.18	145.45	0.313	3.351
22	45.5	45.0	776.2	3889.5	4041.5	383.99	203.71	0.313	2.234
23	47.5	47.0	935.4	2954.0	4976.9	462.79	245.51	0.313	1.675
Avg. Skin			216.4			105.89	56.79	0.313	6.024
Toe			2954.0				10449.36	1.252	1.000

Soil Model Parameters/Extensions		Skin	Toe
Case Damping Factor		0.680	1.614
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	20	
Soil Support Dashpot		2.000	1.700
Soil Support Weight	(kN)	20.00	20.00

Figure 5-16 – CAPWAP final result table

Resource: Cotai Galaxy Hotel & Casino, Area B3 CAPWAP analysis report

Galaxy; Pile: EPC80-3 **Test: 25-Jan-2008**
Blow: 12 **CAPWAP?Ver. 2000-1**
Sol Data (Asia) Limited **Simon Wong (30-Jan-2008)**

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	7668.1	-284.3	39.938	-1.481	92.68	3.3	21.837
2	2.0	7637.0	-266.9	39.776	-1.390	92.09	3.3	21.519
5	5.1	7479.5	-253.0	38.956	-1.318	88.98	3.2	20.543
8	8.1	7352.1	-372.8	38.292	-1.942	84.66	3.1	19.543
11	11.1	7242.5	-543.4	37.721	-2.830	81.76	3.1	18.573
14	14.1	7069.7	-692.9	36.821	-3.609	77.93	3.0	17.612
17	17.2	7038.8	-848.0	36.661	-4.417	75.14	2.9	16.594
20	20.2	6913.8	-992.8	36.010	-5.171	70.48	2.8	15.463
23	23.2	6890.8	-1133.4	35.889	-5.903	64.95	2.7	14.188
26	26.3	6436.5	-1198.7	33.523	-6.243	53.40	2.5	12.851
29	29.3	6134.4	-1311.9	31.950	-6.833	44.75	2.4	11.411
32	32.3	5545.4	-1474.0	28.882	-7.677	35.66	2.3	10.099
35	35.4	5529.9	-1584.7	28.801	-8.254	34.83	2.3	8.976
38	38.4	5436.0	-1645.8	28.313	-8.572	34.23	2.2	8.005
41	41.4	5573.1	-1673.0	29.027	-8.713	33.81	2.1	7.096
42	42.4	5591.8	-1671.7	29.124	-8.707	33.23	2.0	6.796
43	43.5	5812.9	-1675.0	30.276	-8.724	33.17	1.9	6.493
44	44.5	5467.6	-1496.9	28.477	-7.797	29.98	1.8	6.247
45	45.5	5870.1	-1492.6	30.573	-7.774	29.92	1.6	6.017
46	46.5	5141.0	-1238.5	26.776	-6.451	24.98	1.5	5.875
47	47.5	5324.9	-1236.8	27.734	-6.442	19.18	1.4	5.750
Absolute	1.0			39.938			(T =	20.9 ms)
	43.5				-8.724		(T =	63.1 ms)

Figure 5-17 – CAPWAP stress distribution profile

Resource: Cotai Galaxy Hotel & Casino, Area B3 CAPWAP analysis report

5.7 - Usage of dynamic testing methods

Dynamic testing is specified in many ways, depending on the information desired or purpose of the testing. For example, a number of test piles driven at selected locations may be specified. In this application, the test piles are driven at the start of production driving so that the information obtained can be used to establish driving criteria and / or pile order lengths for each substructure unit. Alternatively, testing of production piles on a regular interval may be specified. Production pile testing is usually performed for quality assurance checks on hammer performance, driving stress compliance, pile integrity, and ultimate capacity. Lastly, dynamic testing can be used on projects where it is not specified to troubleshoot problems that arise during construction.

The number of piles that shall be tested on the project depends on the project size, variability of the subsurface conditions, the availability of static load test information, and the reasons for performing the dynamic tests. A higher percentage of piles shall be tested. For example, there are difficult subsurface conditions with an increased risk of the pile damage, or where time dependent soil strength changes are being relied on a significant portion of the ultimate pile capacity.

On small projects, a minimum of two dynamic tests is recommended. On larger projects and small projects with anticipated installation difficulties or significant time dependent capacity, a greater number of piles shall be tested. Dynamically testing one or two piles per substructure location is not unusual in these situations. Regardless of the project size, specifications shall allow the engineer to adjust the number and locations of dynamically tested piles based on design or construction issues that arise.

Restrike dynamic tests shall be performed whenever pile capacity is being evaluated by dynamic test methods. Restikes are commonly specified 24 hours after initial driving. However, in fine grained soils, longer time periods are generally required for the full time dependent capacity changes to occur. Therefore, longer restrike times shall be specified in these soil conditions whenever is possible. On small projects, long restrike durations can present significant construction sequencing problems. At least one longer term restrike shall be performed in these cases. The longer term restrike shall be specified 2 to 6 days after the initial 24 hour restrike, depending on the soil type. A warmed up hammer (from driving or restriking a non-test pile) shall be used whenever restrike tests are performed.

When dynamic testing is performed by a consultant, the requirements for CAPWAP analyses shall be specifically addresses in the dynamic testing specification. On larger projects, CAPWAP analyses are typically performed on 20 to 40% of the dynamic test data obtained from both initial driving and restrike dynamic tests. This percentage typically increases on smaller projects with only a few test piles, or on projects with highly variable subsurface conditions.

It is often contractually convenient to specify that the general contractor retain the services of the dynamic testing firm. However, this can create potential problems since the contractor is then responsible for the agency's quality assurance program. Some agencies have contracted directly with the dynamic testing firm to avoid this potential conflict and many large public owners have purchased the equipment and perform the tests with their own staff.

5.8 - Advantages

Dynamic tests provide information on the complete pile installation process. Test results can be used to estimate pile capacity, to check hammer and drive system performance, to monitor driving stresses, and to assess pile structural integrity.

Many piles can be tested during initial driving or during restrike in one day. This makes dynamic testing an economical and quick testing method. Results are generally available immediately after each hammer blow.

On large projects, dynamic testing can be used to supplement static pile load tests or reduce the overall number of static tests to be performed. Since dynamic tests are more economical than static tests, additional coverage can also be obtained across a project at reduced costs. On small projects where static load tests may be difficult to justify economically, dynamic tests offer a viable construction control method.

Dynamic tests can provide information on pile capacity versus depth, capacity variations between locations, and capacity variations with time after installation through restrike tests. This information can helpful in augmenting the foundation design, when available from design stage test pile programs, or in optimizing pile lengths when used early in construction test programs.

When used as a construction monitoring and quality control tool, dynamic testing can assist in early detection of pile installation problems such as poor hammer performance or high driving stresses. Test results can facilitate the evaluation and solution of these installation problems.

On projects where dynamic testing is not specified and unexpected or unusual driving behavior or pile damage problems develop, dynamic testing offers a quick and economical method of troubleshooting.

Results from dynamic testing and analysis can be used for driving criteria development including wave equation input parameter selection and refinement of wave equation results as described in section 4.5.6.

5.9 - Disadvantages

Dynamic testing to determine the ultimate static pile capacity requires that the driving system mobilize all the soil resistance acting in the pile. Shaft resistance can be mobilized at a fraction of the movement required to mobilize the toe resistance. However, when driving resistances approach 100 blows per 0.25 meter, the full soil resistance is difficult to mobilize at and near the pile toe. In these circumstances, dynamic test capacities tend to produce lower bound capacity estimates unless a larger hammer or higher stroke can be used to increase the pile net penetration per blow.

Dynamic testing estimates of static pile capacity indicate the capacity at the time of testing. Since increases and decreases in the pile capacity with time typically occur due to soil setup / relaxation, restrike tests after an appropriate waiting period are usually required for a better indication of long term pile capacity. This may require an additional move of the pile driving rig for restrike testing.

Larger diameter open ended pipe piles or H-piles which do not bear on rock may behave differently under dynamic and static loading conditions. This is particularly true if a soil plug does not form during driving. In these cases, limited toe bearing resistance develops during the dynamic test. However, under slower static loading conditions, these open section piles may develop a soil plug and therefore a higher pile capacity under static loading conditions. Interpretation of test results by experienced personnel is important in these situations.

CHAPTER 6 - TEST RESULTS

6.1 - Introduction

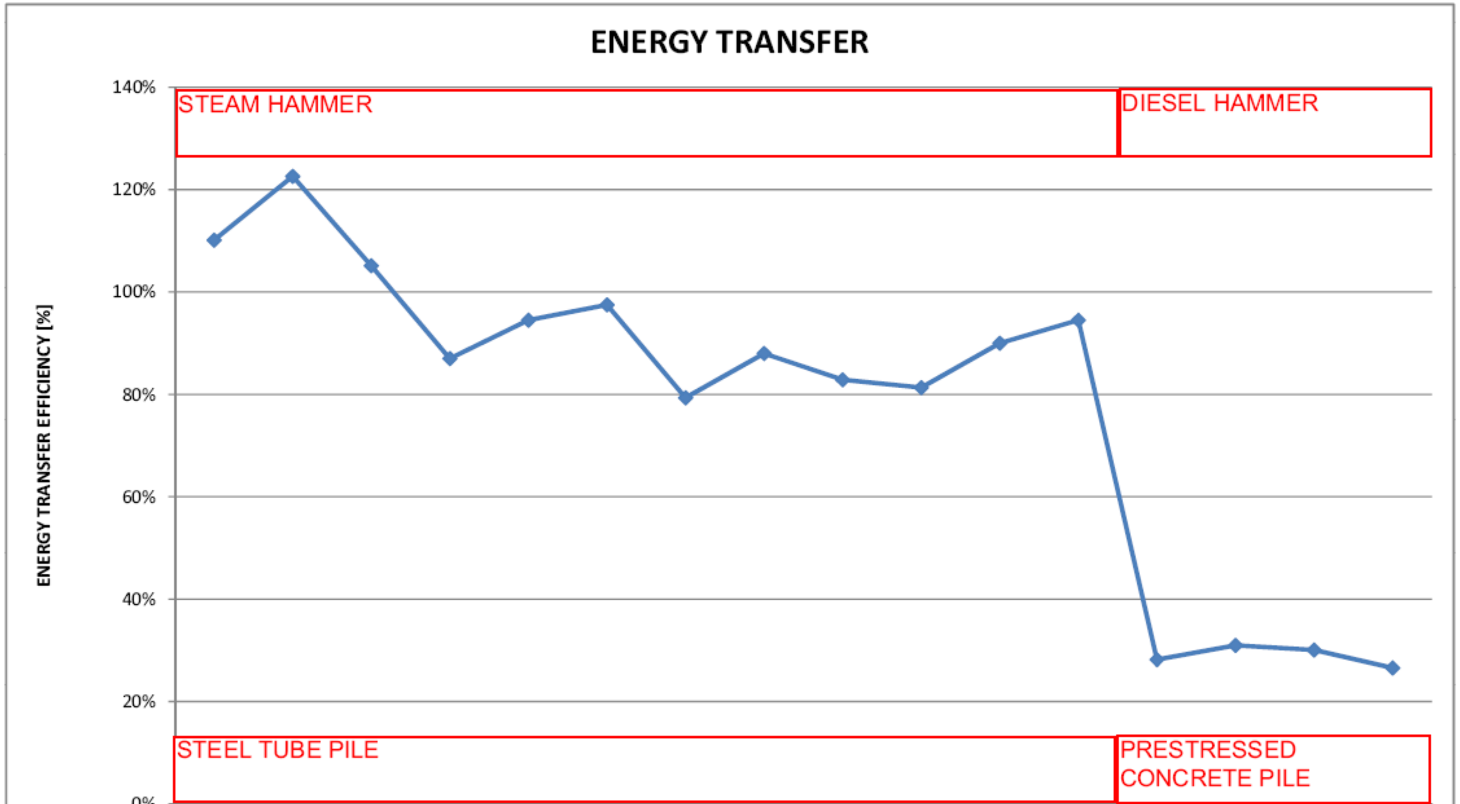
In this chapter, a number of dynamic loading test results from different types of piles, hammers, locations shall be analyzed. West Libya Gas Project – Jetty Works and Cotai Galaxy Hotel & Casion (Macau) will be selected as case study. The test results are presented in tabular form and graphically as follows.

The test results will be analyzed in following aspects:

- Energy transfer (hammer performance)
- Driving stress
- Pile integrity
- Static capacity

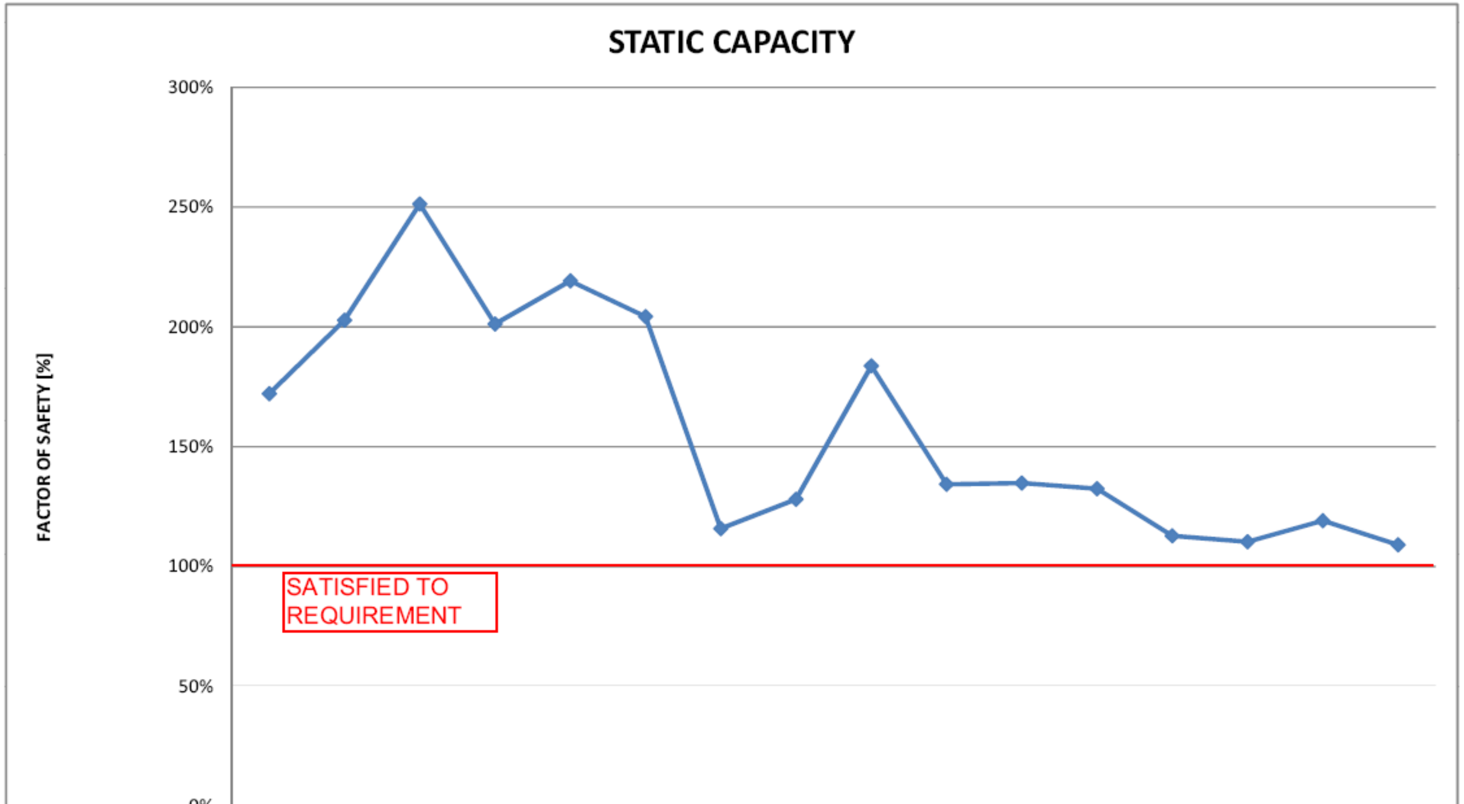
Pile no.	Project	Type of pile	Type of hammer					Static capacity			Compressive stress			Integrity	
			Type	Model	Output maximum energy	Measured	Efficiency	Required	Measured	Factor of safety	Allowable	Measured	Ratio	Factor	Severity of damage
					[kNm]	[kNm]	[%]	[kN]	[kN]	[%]	[MPa]	[MPa]	[%]	[%]	
TP 1-3	West Libya	Steel tube	Steam	IHC S-150	150	165.2	110%	12000	20660	172%	323.1	202	62.5%	100%	Undamaged
TP 1-1	West Libya	Steel tube	Steam	IHC S-200	200	245.2	123%	12000	24340	203%	323.1	257	79.5%	100%	Undamaged
TP 1-2	West Libya	Steel tube	Steam	IHC S-200	200	210.3	105%	12000	30160	251%	323.1	223	69.0%	100%	Undamaged
TP 3-3	West Libya	Steel tube	Steam	IHC S-200	200	174	87%	12000	24150	201%	323.1	205	63.4%	100%	Undamaged
TP 3-2	West Libya	Steel tube	Steam	IHC S-200	200	189	95%	12000	26310	219%	323.1	222	68.7%	100%	Undamaged
TP 5-2	West Libya	Steel tube	Steam	IHC S-200	200	195	98%	12000	24520	204%	323.1	218	67.5%	100%	Undamaged
TP 3-1	West Libya	Steel tube	Steam	IHC S-150	150	119	79%	12000	13900	116%	323.1	189	58.5%	100%	Undamaged
TP 4-1	West Libya	Steel tube	Steam	IHC S-200	200	176	88%	12000	15370	128%	323.1	202	62.5%	100%	Undamaged
TP 5-1	West Libya	Steel tube	Steam	IHC S-200	200	165.7	83%	12000	22050	184%	323.1	208.5	64.5%	100%	Undamaged
TP 4-2	West Libya	Steel tube	Steam	IHC S-150	150	122	81%	12000	16120	134%	323.1	176	54.5%	100%	Undamaged
TP 4-3	West Libya	Steel tube	Steam	IHC S-200	200	180	90%	12000	16180	135%	323.1	213	65.9%	100%	Undamaged
TP 6-2	West Libya	Steel tube	Steam	IHC S-200	200	189	95%	12000	15900	133%	323.1	227	70.3%	100%	Undamaged
EPC 80-3	Macau	Prestreeed concrete pile	Diesel	Delmag D80-23	288	81.2	28%	7000	7897	113%	64	46.6	72.8%	95%	Slightly damaged
EPC 81A-3	Macau	Prestreeed concrete pile	Diesel	Delmag D80-23	288	89.2	31%	7000	7723	110%	64	48.2	75.3%	89%	Slightly damaged
EPC 84-6	Macau	Prestreeed concrete pile	Diesel	Delmag D80-23	288	86.6	30%	7000	8340	119%	64	43.2	67.5%	88%	Slightly damaged
EPC 85-1	Macau	Prestreeed concrete pile	Diesel	Delmag D80-23	288	76.5	27%	7000	7633	109%	64	48	75.0%	90%	Slightly damaged

Table 6-1 – Summary of dynamic loading test result



	TP 1-3	TP 1-1	TP 1-2	TP 3-3	TP 3-2	TP 5-2	TP 3-1	TP 4-1	TP 5-1	TP 4-2	TP 4-3	TP 6-2	EPC 80-3	EPC 81A-3	EPC 84-6	EPC 85-1
Energy Transfer	110%	123%	105%	87%	95%	98%	79%	88%	83%	81%	90%	95%	28%	31%	30%	27%

Figure 6-1 – Graphical presentation of energy transfer



	TP 1-3	TP 1-1	TP 1-2	TP 3-3	TP 3-2	TP 5-2	TP 3-1	TP 4-1	TP 5-1	TP 4-2	TP 4-3	TP 6-2	EPC 80-3	EPC 81A-3	EPC 84-6	EPC 85-1
STATIC CAPACITY	172%	203%	251%	201%	219%	204%	116%	128%	184%	134%	135%	133%	113%	110%	119%	109%

Figure 6-2 – Graphical presentation of static capacity

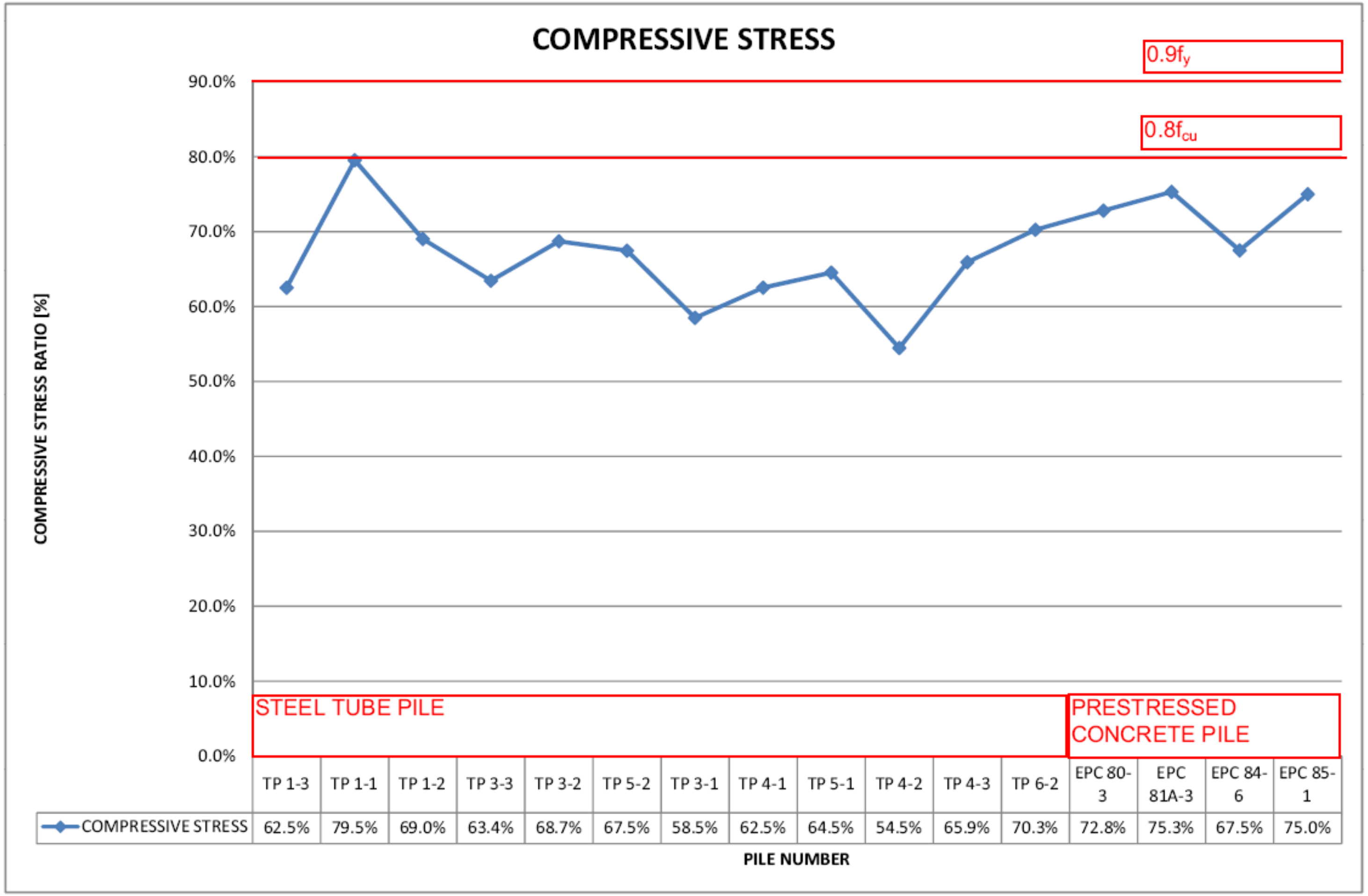


Figure 6-3 – Graphical presentation of compressive stress

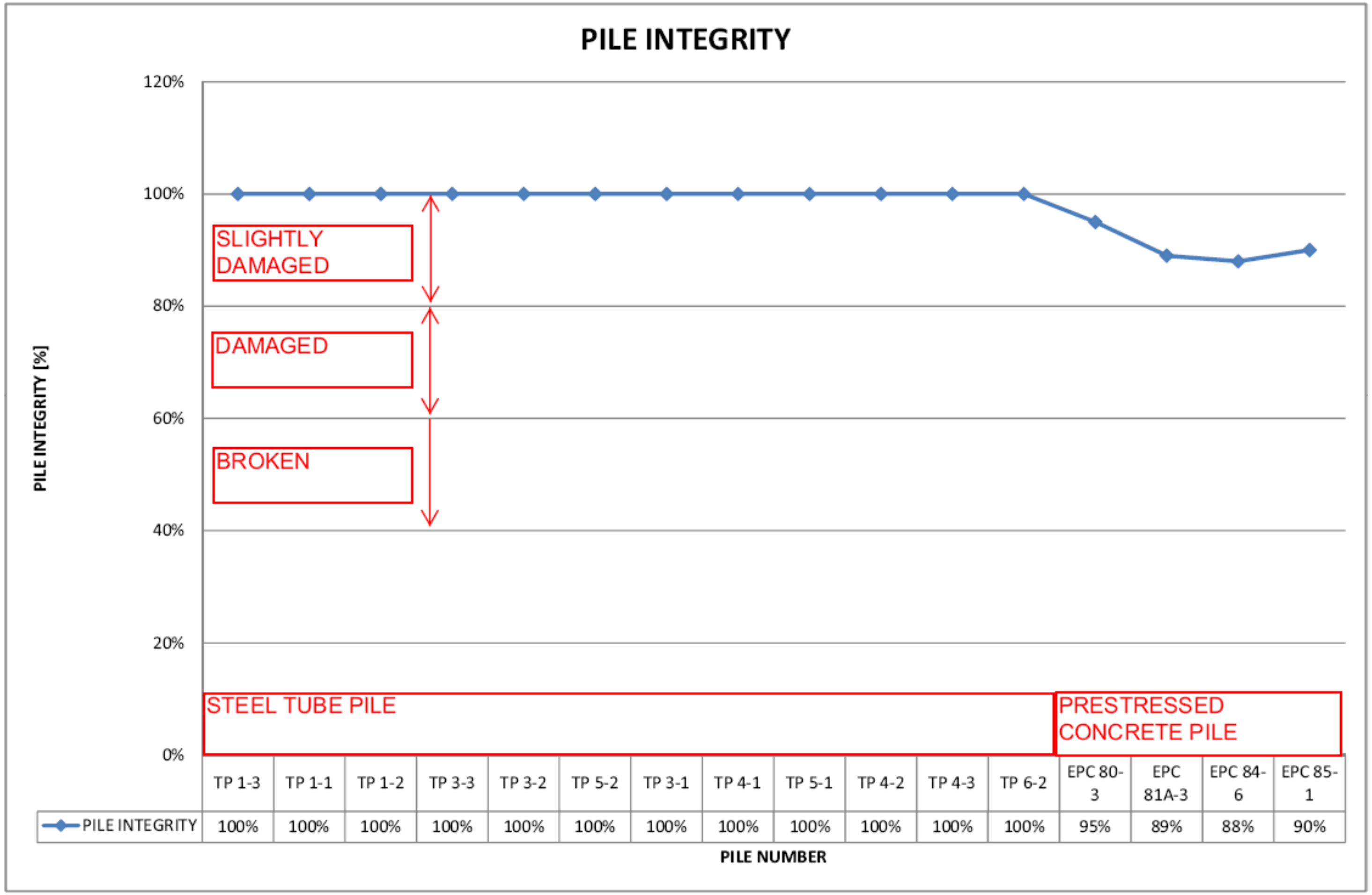


Figure 6-4 – Graphical presentation of pile integrity

6.2 - Interpretation

6.2.1 - Energy transfer

From the energy transfer plot (Figure 6-1), the steel piles are stroke by steam hammer (model IHC) and the prestressed concrete piles are stroke by diesel hammer (Demag). The average energy transfer of steam hammer is approximate 90%. It is a little lower than the specification mentioned maximum energy output. The average energy transfer of diesel hammer is approximate 30%. It is far from the maximum energy output which is mentioned in hammer specification.

The energy transfer of steel pile TP1-1, TP1-2 and TP1-3 are recorded over 100% energy input. This phenomenon is normal and will be explained in Chapter 7.

6.2.2 - Driving stress

Figure 6-3 is showing the driving stress (compressive stress) of steel piles and concrete piles with steam hammer and diesel hammer. The average driving stress of steel pile is in range of 60% to 70% of the allowable driving stress ($323.1N/mm^2$). The average driving stress of concrete pile is in range of 70% to 80% of the allowable driving stress ($64N/mm^2$).

The driving stress in steel piles and concrete piles are below the maximum compressive stress, which is $0.9f_y$ and $0.8f_{cu}$. Therefore, no overstressing is recorded during restrike test.

6.2.3 - Pile integrity

Figure 6-4 is presenting the integrity of steel pile and concrete pile. The integrity of steel piles is 100% and it is classified as no damaged. However, the integrity of concrete pile is fallen into range of 80% to 100% and therefore classified as slightly damaged.

6.2.4 - Pile capacity

From the plot figure 6-2, the static capacity of both steel and concrete pile is fulfilling to the requirement. Over 50% of steel piles are exceeding the required static capacity 50% and are achieving to 150%. The average static capacity of concrete piles is just over 10% of the required static capacity.

CHAPTER 7 - DISCUSSION AND CONCLUSION

7.1 - Discussion

7.1.1 - Pile integrity

Apart from determining the static capacity of piles, the pile integrity is also a critical item to be concerned during driving. The pile is driven into the ground and becomes ‘out of sight’. In order to prevent ‘out of mind’, Rausche & Goble (1979) suggested the use of a damage classification factor, β_Z, which is defined in terms of changes in impedance.

$$\beta_z = \frac{Z_2}{Z_1} \tag{7-1}$$

where Z_2 = Pile impedance above a given level where there is a significant change in impedance

Z_1 = Pile impedance below the same given level

Impedance, Z, is defined as equation 5-3.

The tentative classification scheme proposed by Rausche & Goble (1979) is reproduced in Table 5-2. This simplified method is related to the extent of pile cross-section that is left after the damage, and is based on the tacit assumption that the soil resistance immediately below the point of damage is negligible.

From the graph figure 6-4, the integrity of all steel tube piles are 100% and therefore classified undamaged. However, the integrity of prestressed concrete piles are all within 0.8 – 1.0. It belongs to slightly damaged.

With reference to the graph – compressive stress, the stresses on pile are under the limits of driving stress $0.8f_{cu}$. The pile damaged by excessive compressive stress is therefore not established.

Retracing to the pile integrity equation, the pile integrity is the ratio of pile impedance. The pile impedance is in term of section modulus, cross section area and the velocity of stress wave. From the equation, the pile impedance is governed by the section modulus and cross section area. Therefore, it is suspected of the workmanship during casting the concrete or the collapse of soil in pile shaft location during withdrawing the steel tube after pouring concrete. The integrity of pile is varied due to suddenly change of section modulus or cross section area.

In this case, further physical coring test is necessary to provide samples for visual examination and for compression testing.

7.1.2 - Stress on pile

Damage to piles during driving is visible only near the pile head, but the shaft and toe may also be damaged. Failure due to excessive compressive stress most commonly occurs at the pile head. Tensile stresses are caused by reflection of the compressive waves at a free end and may arise when the ground resistance is low or when the head conditions result in hammer rebound.

The driving stresses must not exceed the limiting values that will cause damage to the pile. The following limits on driving stresses suggested by BS EN 12699:2001 clause 7.7.2.1 and 7.7.3.1 are described the driving stress limits on both steel and concrete pile.

Pile type		Maximum compressive stress	Maximum tensile force
Steel piles		$\leqq 0.9f_y$	-
Prefabricated concrete piles (including prestressed piles)		$\leqq 0.8f_{cu}$	$\leqq 0.9f_yA_s$ – Presstressing force
Notes:	(1) fy is the yield stress of steel, As is the area of steel reinforcement and fcu is the specified grade strength of concrete. (2) If driving stress is actually monitored during driving, the limits can be increased by 10% and 20% for prefabricated concrete piles and steel piles respectively.		

Table 7-1 - Limits on driving stress (BSI, 2001)

From the graph compressive stress, both steel tube piles and prestressed concrete piles are under the driving stress limits. Therefore, no piles are damaged due to overstressing during restrike test.

With reference to the energy transfer plot, the energy recorded by striking concrete piles is relatively lower than the maximum energy output. However, the result of driving stress approaches to the limited value. Obviously, controlling the hammer energy transferring into the pile can also be controlling the driving stress of the pile.

7.1.3 - Hammer performance

From the piles which are driven in West Libya Gas Project, 2 different kinds of hammer with various driving force were used. Under normal conditions, the dynamic resistance is created by the ground / soil resistance to driving which is a combination of end bearing resistance and skin friction resistance. The change in hammers would normally have negligible effect on the driving resistance. However in this particular pile test, the S-150 hammer is not driving the pile efficiently and several blows are required to drive the pile by 5 mm. Normally, a pile is considered to be fully mobilized if the movement per blow is at 5 mm. It is therefore considered that the pile is only being partially mobilized by S-150 on each blow and therefore the value of dynamic resistance may be a slight underestimation. When the S-200 hammer is then used, the pile is fully mobilized on each blow hence a more accurate and higher value of driving resistance is obtained.

From the table shown, some of the energy transfer efficiency are over 100% no matter what hammer to be used. Basically, it is not a reasonable phenomenon in the driving process because it implies no energy loss during pile driving. Oppositely, an additional energy is infused into the pile.

The law of conservation of energy states that the total amount of energy in an isolated system remains constant. A consequence of this law is that energy cannot be created nor destroyed. The only thing that can happen *with energy in an isolated system is that it can change form, for instance kinetic energy can become thermal energy.*

Albert Einstein's theory of relativity shows that energy can be converted to mass (rest mass) and mass converted to energy. Therefore, neither mass nor pure energy are conserved separately, as it was understood in pre-relativistic physics. Today, conservation of "energy" refers to the conservation of the total mass-energy, which includes energy of the rest mass. Therefore, in an isolated system, mass and "pure energy" can be converted to one another, but the total amount of energy (which includes the energy of the mass of the system) remains constant. [http://en.wikipedia.org/wiki/Conservation_of_energy]

In this case, it is probably that the remaining energy by means of vibration from prior blow has not been totally used up. The second blow is then hitting on the pile head incessantly. Hence the 'new' energy will combine with the remaining energy and therefore generate a large energy. This phenomenon is likely to the motion of wave.

The rating of piling hammer is based on the gross energy per blow. However, different types of hammers have differing efficiencies in terms of the actual energy transmitted through the pile being driven. The range of typical efficiencies of different types of hammers is shown in table 7-2.

If the driving speed slows down to let the remaining energy 'used up', this phenomenon is not occurred again. Moreover, the energy transfer ratio of such piles are fulfilling into the below typical energy transfer table.

Type of hammer		Typical energy transfer ratio
Drop hammers		0.45 – 0.6
Hydraulic hammers		0.7 - 1
Notes	(1) Energy transfer ratio corresponds to the ratio of actual energy transferred to the pile to the rated capacity of the hammer. (2) Actual amount of energy transferred to the pile is best determined by dynamic pile testing. (3) The above are based on general experience in Hong Kong	

Table 7-2 - Typical energy transfer ratio of pile hammers

4 numbers of prestressed concrete pile which are driven by diesel hammer have extremely low energy transfer ratio. According to the table, the efficiency of diesel hammer should be at least 70%. Obviously, the piles are driven under inefficiency. However, referring to the graph – compressive stress, the stresses are near to the yield stress limits $0.8f_{cu}$. It

is believed that the piles may be failed if the energy of hammer is increased in term of blow rate and / or impact force of ram.

7.1.4 - Pile capacity

The primary objective of the dynamic loading test is determining the static capacity of the driven pile. The actual pile bearing capacity whether is fulfilling to the design requirement is the most concerned. From the graph shown, all steel tube piles and prestressed concrete piles are over the required pile capacity.

However, it is not presented the 'ultimate' bearing capacity is obtained. This is because the pile has not been driven in fail condition. Normally, a pile is considered to be failed (fully mobilized) if the movement of pile is over 5 mm per blow. The below table is listed the acceptance criteria of pile bearing capacity via pile movement.

Movement of pile	**Description**
Displacement of pile more than 5 mm per blow	Unacceptable (excessive pile movement and pile cannot support such driving force)
Displacement equal to 5 mm per blow	Acceptable (pile bearing capacity is achieving to ultimate bearing capacity)
Displacement of pile less than 5 mm per blow	Acceptable (pile bearing capacity is underestimate)

Table 7-3 - Acceptance criteria of driven pile via pile displacement

It is therefore considered that the pile is only being partially mobilized and therefore the value of dynamic resistance may be a slight underestimation. When the driving force increase, the pile is more fully mobilized on each blow hence a more accurate and higher value of driving resistance is obtained.

Although increasing the driving force can obtain more accurate and higher value of driving resistance, it will also increase the risk of over driving stress of pile and cause to pile damaged.

7.2 - Conclusion

The main conclusions that can be drawn from the tests are as follows:

- The measured bearing capacity of piles is higher than the required capacity but the ultimate bearing capacity has not been obtained due to the pile not driven in fail condition.

- Energy transfer is depending on the blow rate and the ram drop height and size. Higher energy transfer can be gained either blow rate or hammer size increased.

- The driving stress can also be controlled by energy transfer. Increasing the energy transfer is implied that the compressive will then be increased relatively.

- Limitation of driving stress shall be determined before driving, otherwise, the pile is probably overstressing and damaged.

- Comparison with the integrity of steel and concrete pile, all steel piles are classified no damages and all concrete piles are classified slightly damaged. It is presented that driving concrete pile is easier damage than driving steel pile.

- During pile driving, the pile integrity is the most critical point to be monitored. The pile integrity is also related with driving stress and energy transfer. The table shown below is well described to relationship between pile capacity, integrity, driving stress and energy transfer.

	Pile capacity ↑	Energy transfer ↑	Driving stress ↑	Integrity ↑
Pile capacity		↑	↑	↓
Energy transfer	↑		↑	↓
Driving stress	↑	↑		↓
Integrity	↓	↓	↓	

Table 7-4 – Relationship between pile capacity, Energy transfer, Driving stress and Integrity

Increasing the hammer energy transfer can achieve to gain higher pile capacity. However, due to the energy transfer increased, the driving stress will then be risen up. As the result, the risk of pile damages is also increased.

Oppositely, in order to prevent damaging to pile, decreasing the driving stress by means of lowering the energy transfer of hammer can achieve. Although the risk of pile damage is reduced, lower pile capacity is also gained.

7.3 - Recommendation

The result of analysis is not very reliable because a few pile driving test data is collected. A large number of test data is necessary to increase the reliability and accuracy.

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APPENDIX A – PROJECT SPECIFICATION

University of Southern Queensland

FACULTY OF ENGINEERING AND SURVEYING

ENG4111/4112 Research Project

PROJECT SPECIFICATION

For: Yu Yu Wah

TOPIC: PILE DRIVING ANALYSIS VIA DYNAMIC LOADING TEST

SUPERVISORS: Jim Shiau

PROJECT AIM: Research on the Dynamic loading test and using such data to determine / correlation the tolerance of pile design formula.

PROGRAMME: **Issue A, 24th March, 2007**

1. Research the dynamic loading test result in various soli strata which are using PDI apparatus with couple of strain gauges and accelerometers.
2. Compare the result with bore hole records and evaluate the characteristic of each soil layer.
3. Determinate the pile integrity, compressive / tensile stress on pile, bearing capacity and efficiencies of hammer performance.
4. Compare the test result with static loading test and evaluate the accuracy.
5. Determine the cause of pile defect / damage.

AGREED:

____________________(Student)____(Supervisors)

24/03/2008 ____/____/____

Examiner/Co-examiner:____

APPENDIX B – RAW DATA

TEST RESULT - DYNAMIC PILE TEST (TP1-1)

CLIENT: BESIX S.A. – LIBYA BRANCH

PROJECT: WEST LIBYA GAS PROJECT – JETTY WORKS

DATE: 1/11/2002

TYPE OF TEST: ☑ RESTRIKE ☐ END OF DRIVE

EQUIPMENT:

PDA	-	ANGLE METER	-	MEASURING TAPE	-

		[F1]		[F2]
STRAIN GAUGE	EQU. NO.	-	EQU. NO.	-
	CAL	-	CAL	-
		[A1]		[A2]
ACCELEROMETER	EQU. NO.	-	EQU. NO.	-
	CAL	-	CAL	-

HAMMER PROPERTIES:

HAMMER TYPE	☑ HYDRAULIC	☐ DROP HAMMER	☐ SPECIFIED	
TYPE OF CUSHION	☐ PLYWOOD	☐ RUBBER	☑ SPECIFIED	NONE
SERIAL NO.	-	WEIGHT OF RAM	- [ton]	

PILE PROPERTIES:

PILE NO.	TP1-1	L_t	32.55 [m]	ELASTIC MODULUS	210000 [MPa]
TYPE	Ø1422 STEEL PIPE	L_e	28.30 [m]	YIELD STRENGTH	355 [MPa]
INCLINATION	- []	L_p	17.60 [m]	REQUIRE CAPACITY	12000 [kN]
AREA	0.110 [m^2]	GRADE	X52	WAVESPEED	5172 [m/s]
COATING	NONE	IMPEDANCE	[kNs/m]	SPECIFIC WEIGHT	7.85 [kN/m^3]

TEST RESULTS:

BLOW NO.		3240				
FINAL SET		-	[mm / BLOW]			
NEAREST BOREHOLE RECORD		-				
DATE OF LAST HAMMERING		1/11/2002				
CASE STATIC	J_c = 0.0	-	[kN]	J_c = 0.5	-	[kN]
BEARING	J_c = 0.1	20660	[kN]	J_c = 0.6	-	[kN]
CAPACITY WITH VARIOUS	J_c = 0.2	-	[kN]	J_c = 0.7	-	[kN]
DAMPING	J_c = 0.3	-	[kN]	J_c = 0.8	-	[kN]
FACTORS	J_c = 0.4	-	[kN]	J_c = 0.9	-	[kN]
MAX. FORCE IN SENSOR LEVEL		71.71	[kN]			
MAX. COMPRESSIVE STRESS		202	[MPa]			
MAX. TENSILE STRESS		-	[MPa]			
MAX. ENERGY TRANSFERRED		165.2	[kNm]			
HAMMER DROP HEIGHT		-	[m]			
HAMMER RATED ENERGY		150	[kNm]			
ENERGY TRANSFERRED		110%	[%]			
PILE INTEGRITY		100	[%]			

PILE DAMAGED:

PILE DAMAGED AT LEVEL	- [m]	FROM THE LEVEL OF GAUGES	- [m]

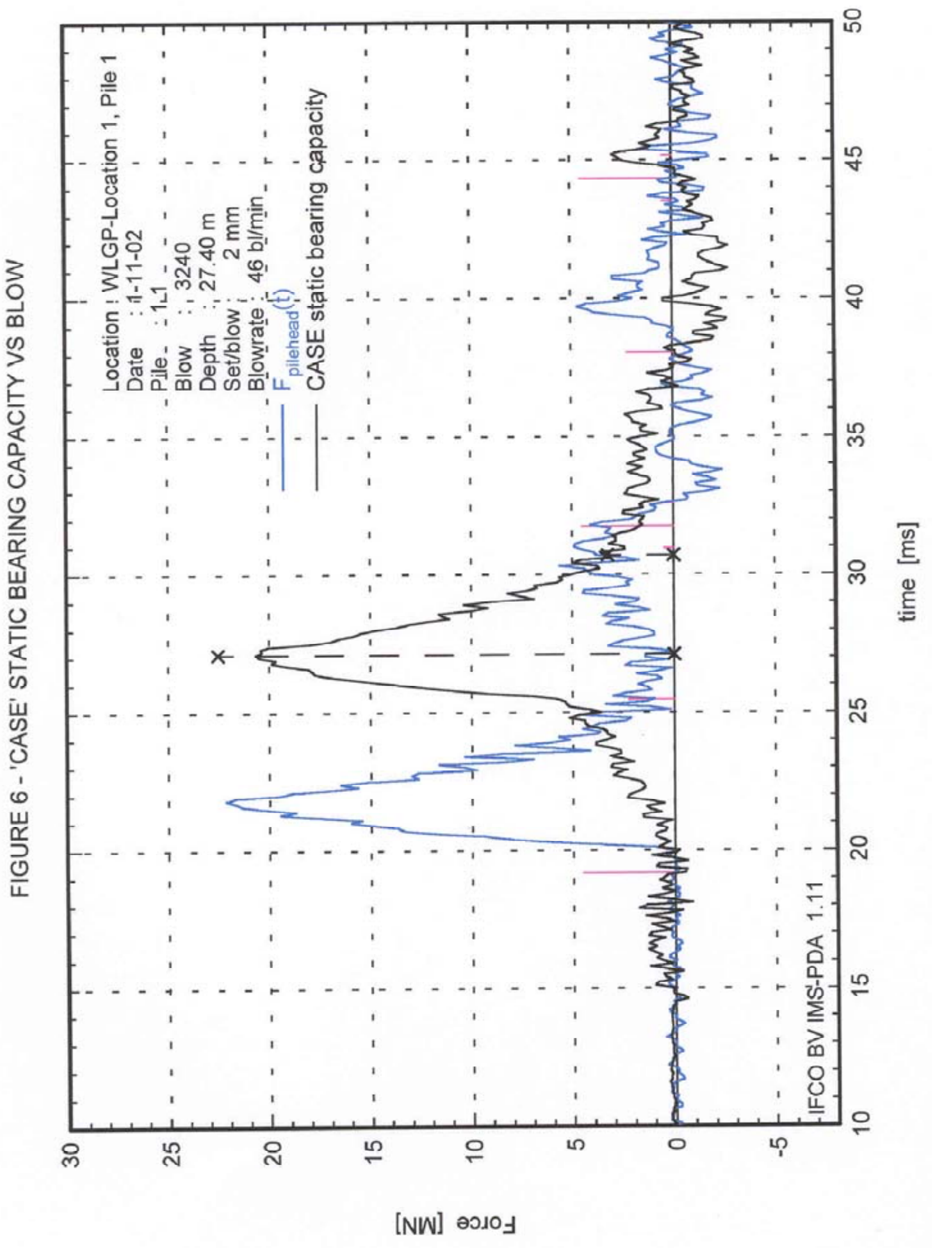


FIGURE 6 - 'CASE' STATIC BEARING CAPACITY VS BLOW
Location : WLGP-Location 1, Pile 1
Date : 1-11-02
Pile : 1L1
Blow : 3240
Depth : 27.40 m
Set/blow : 2 mm
Blowrate : 46 bl/min
F pilehead (t)
CASE static bearing capacity
IFCO BV IMS-PDA 1.11
time [ms]
Force [MN]
10
15
20
25
30
35
40
45
50
30
25
20
15
10
5
0
-5

TEST RESULT - DYNAMIC PILE TEST (TP1-2)

CLIENT: BESIX S.A. – LIBYA BRANCH

PROJECT: WEST LIBYA GAS PROJECT – JETTY WORKS

DATE: 2/11/2002 *TYPE OF TEST:* ☑ RESTRIKE ☐ END OF DRIVE

EQUIPMENT:

PDA	-	ANGLE METER	-	MEASURING TAPE	-

		[F1]		[F2]
STRAIN GAUGE	EQU. NO.	-	EQU. NO.	-
	CAL	-	CAL	-
		[A1]		[A2]
ACCELEROMETER	EQU. NO.	-	EQU. NO.	-
	CAL	-	CAL	-

HAMMER PROPERTIES:

HAMMER TYPE	☑ HYDRAULIC	☐ DROP HAMMER	☐ SPECIFIED	
TYPE OF CUSHION	☐ PLYWOOD	☐ RUBBER	☑ SPECIFIED	NONE
SERIAL NO.	-	WEIGHT OF RAM	- [ton]	

PILE PROPERTIES:

PILE NO.	TP1-2	L_t	32.55 [m]	ELASTIC MODULUS	210000 [MPa]
TYPE	Ø1422 STEEL PIPE	L_e	28.30 [m]	YIELD STRENGTH	355 [MPa]
INCLINATION	- []	L_p	17.60 [m]	REQUIRE CAPACITY	12000 [kN]
AREA	0.110 [m²]	GRADE	X52	WAVE SPEED	5172 [m/s]
COATING	NONE	IMPEDANCE	[kNs/m]	SPECIFIC WEIGHT	7.85 [kN/m³]

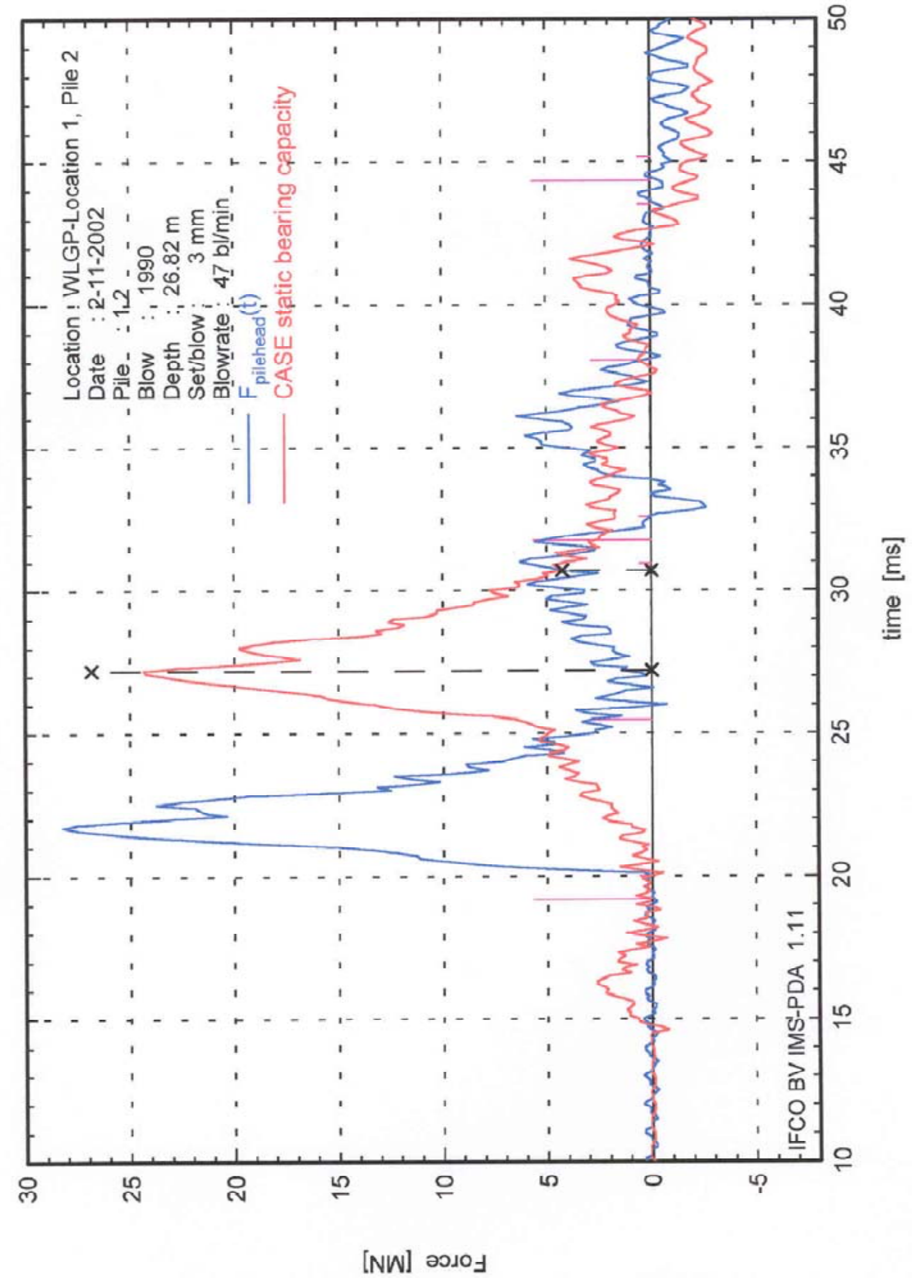
TEST RESULTS:

BLOW NO.		1990			
FINAL SET		- [mm / BLOW]			
NEAREST BOREHOLE RECORD		-			
DATE OF LAST HAMMERING		2/11/2002			
CASE STATIC BEARING CAPACITY WITH VARIOUS DAMPING FACTORS	$J_c = 0.0$	- [kN]	$J_c = 0.5$	- [kN]	
	$J_c = 0.1$	24340 [kN]	$J_c = 0.6$	- [kN]	
	$J_c = 0.2$	- [kN]	$J_c = 0.7$	- [kN]	
	$J_c = 0.3$	- [kN]	$J_c = 0.8$	- [kN]	
	$J_c = 0.4$	- [kN]	$J_c = 0.9$	- [kN]	
MAX. FORCE IN SENSOR LEVEL		91.235 [kN]			
MAX. COMPRESSIVE STRESS		257 [MPa]			
MAX. TENSILE STRESS		- [MPa]			
MAX. ENERGY TRANSFERRED		245.2 [kNm]			
HAMMER DROP HEIGHT		- [m]			
HAMMER RATED ENERGY		200 [kNm]			
ENERGY TRANSFERRED		123% [%]			
PILE INTEGRITY		100 [%]			

PILE DAMAGED:

PILE DAMAGED AT LEVEL - [m] FROM THE LEVEL OF GAUGES - [m]

FIGURE 6 - 'CASE' STATIC BEARING CAPACITY VS BLOW



TEST RESULT - DYNAMIC PILE TEST (TP1-3)

CLIENT: BESIX S.A. – LIBYA BRANCH

PROJECT: WEST LIBYA GAS PROJECT – JETTY WORKS

DATE: 9/12/2002

TYPE OF TEST: ☑ RESTRIKE ☐ END OF DRIVE

EQUIPMENT:

PDA	-	ANGLE METER	-	MEASURING TAPE	-
		[F1]		[F2]	
STRAIN GAUGE	EQU. NO.	-	EQU. NO.	-	
	CAL	-	CAL	-	
		[A1]		[A2]	
ACCELEROMETER	EQU. NO.	-	EQU. NO.	-	
	CAL	-	CAL	-	

HAMMER PROPERTIES:

HAMMER TYPE	☑ HYDRAULIC	☐ DROP HAMMER	☐ SPECIFIED	
TYPE OF CUSHION	☐ PLYWOOD	☐ RUBBER	☑ SPECIFIED	NONE
SERIAL NO.	-	WEIGHT OF RAM	- [ton]	

PILE PROPERTIES:

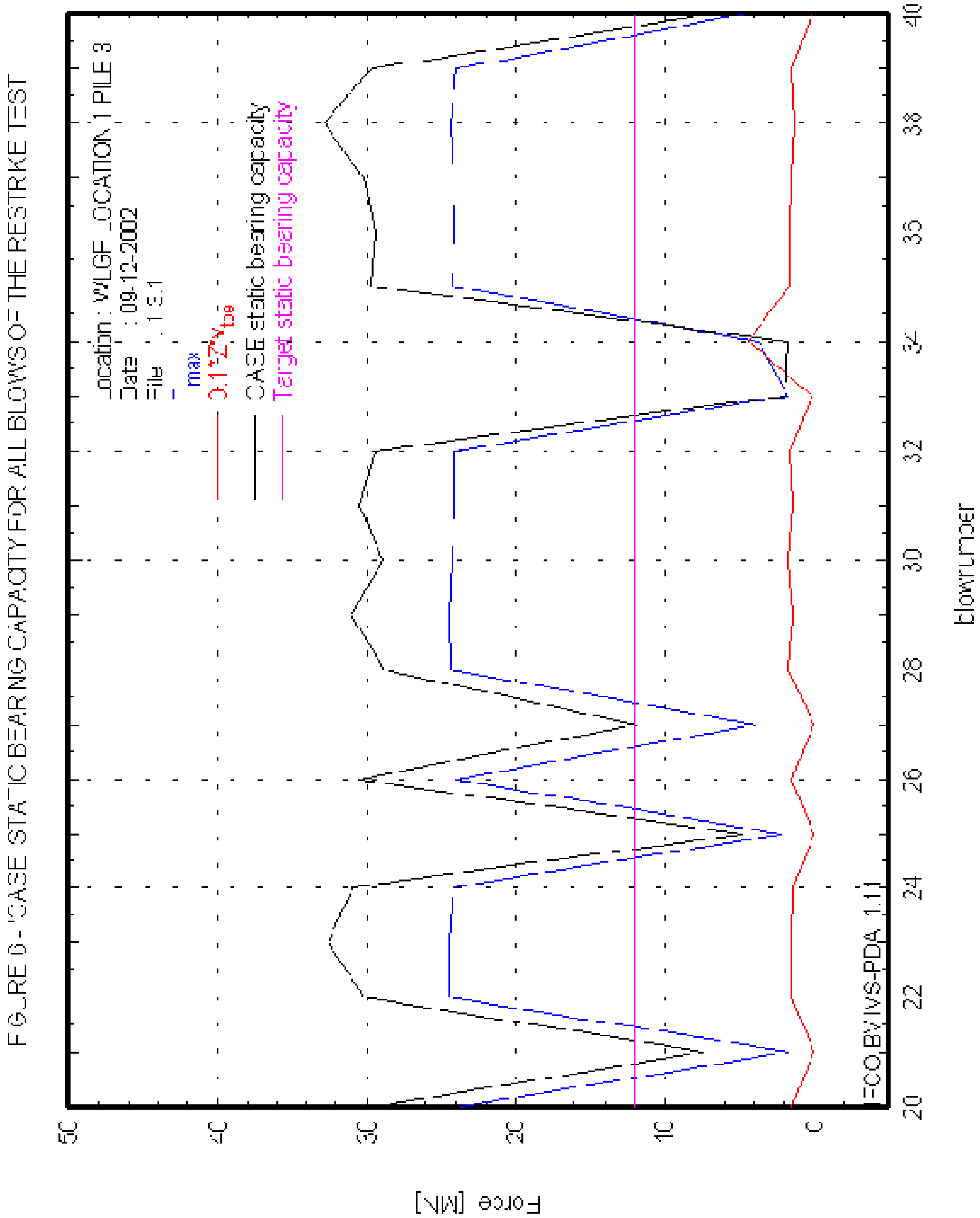
PILE NO.	TP1-3	L_t	32.55 [m]	ELASTIC MODULUS	210000 [MPa]
TYPE	Ø1422 STEEL PIPE	L_e	28.30 [m]	YIELD STRENGTH	355 [MPa]
INCLINATION	- []	L_p	17.50 [m]	REQUIRE CAPACITY	12000 [kN]
AREA	0.110 [m²]	GRADE	X52	WAVE SPEED	5172 [m/s]
COATING	NONE	IMPEDANCE	4.06E+03 [kNs/m]	SPECIFIC WEIGHT	7.85 [kN/m³]

TEST RESULTS:

BLOW NO.		22		
FINAL SET		- [mm / BLOW]		
NEAREST BOREHOLE RECORD		-		
DATE OF LAST HAMMERING		2/11/2002		
CASE STATIC BEARING CAPACITY WITH VARIOUS DAMPING FACTORS	J_c = 0.0	- [kN]	J_c = 0.5	- [kN]
	J_c = 0.1	30100 [kN]	J_c = 0.6	- [kN]
	J_c = 0.2	- [kN]	J_c = 0.7	- [kN]
	J_c = 0.3	- [kN]	J_c = 0.8	- [kN]
	J_c = 0.4	- [kN]	J_c = 0.9	- [kN]
MAX. FORCE IN SENSOR LEVEL		79.165 [kN]		
MAX. COMPRESSIVE STRESS		223 [MPa]		
MAX. TENSILE STRESS		- [MPa]		
MAX. ENERGY TRANSFERRED		210.3 [kNm]		
HAMMER DROP HEIGHT		- [m]		
HAMMER RATED ENERGY		200 [kNm]		
ENERGY TRANSFERRED		105% [%]		
PILE INTEGRITY		100 [%]		

PILE DAMAGED:

PILE DAMAGED AT LEVEL - [m] FROM THE LEVEL OF GAUGES - [m]


F G_RE D - 'CASE STA'IC BEAR NG CAPACITY FOR ALL BLOWS OF THE RESTRKE TEST
_ocation : WLGF _OCATION 1 PILE 3
Date : 09-12-2002
max
CASE static bearing capacity
Target static bearing capacity
blownumber
Force [MN]

TEST RESULT - DYNAMIC PILE TEST (TP3-3)

CLIENT: BESIX S.A. – LIBYA BRANCH

PROJECT: WEST LIBYA GAS PROJECT – JETTY WORKS

DATE: 18/11/2002

TYPE OF TEST: ☑ RESTRIKE ☐ END OF DRIVE

EQUIPMENT:

PDA - ANGLE METER - MEASURING TAPE -

		[F1]		[F2]
STRAIN GAUGE	EQU. NO.	-	EQU. NO.	-
	CAL	-	CAL	-
		[A1]		[A2]
ACCELEROMETER	EQU. NO.	-	EQU. NO.	-
	CAL	-	CAL	-

HAMMER PROPERTIES:

HAMMER TYPE ☑ HYDRAULIC ☐ DROP HAMMER ☐ SPECIFIED

TYPE OF CUSHION ☐ PLYWOOD ☐ RUBBER ☑ SPECIFIED NONE

SERIAL NO. - WEIGHT OF RAM - [ton]

PILE PROPERTIES:

PILE NO.	TP3-3	L_1	41 [m]	ELASTIC MODULUS	210000 [MPa]
TYPE	914x22 STEEL PIPE	L_e	36.75 [m]	YIELD STRENGTH	355 [MPa]
INCLINATION	- []	L_p	23.01 [m]	REQUIRE CAPACITY	12000 [kN]
AREA	0.101 [m²]	GRADE	X52	WAVESPEED	5172 [m/s]
COATING	NONE	IMPEDANCE	4.06E+03 [kNs/m]	SPECIFIC WEIGHT	7.85 [kN/m³]

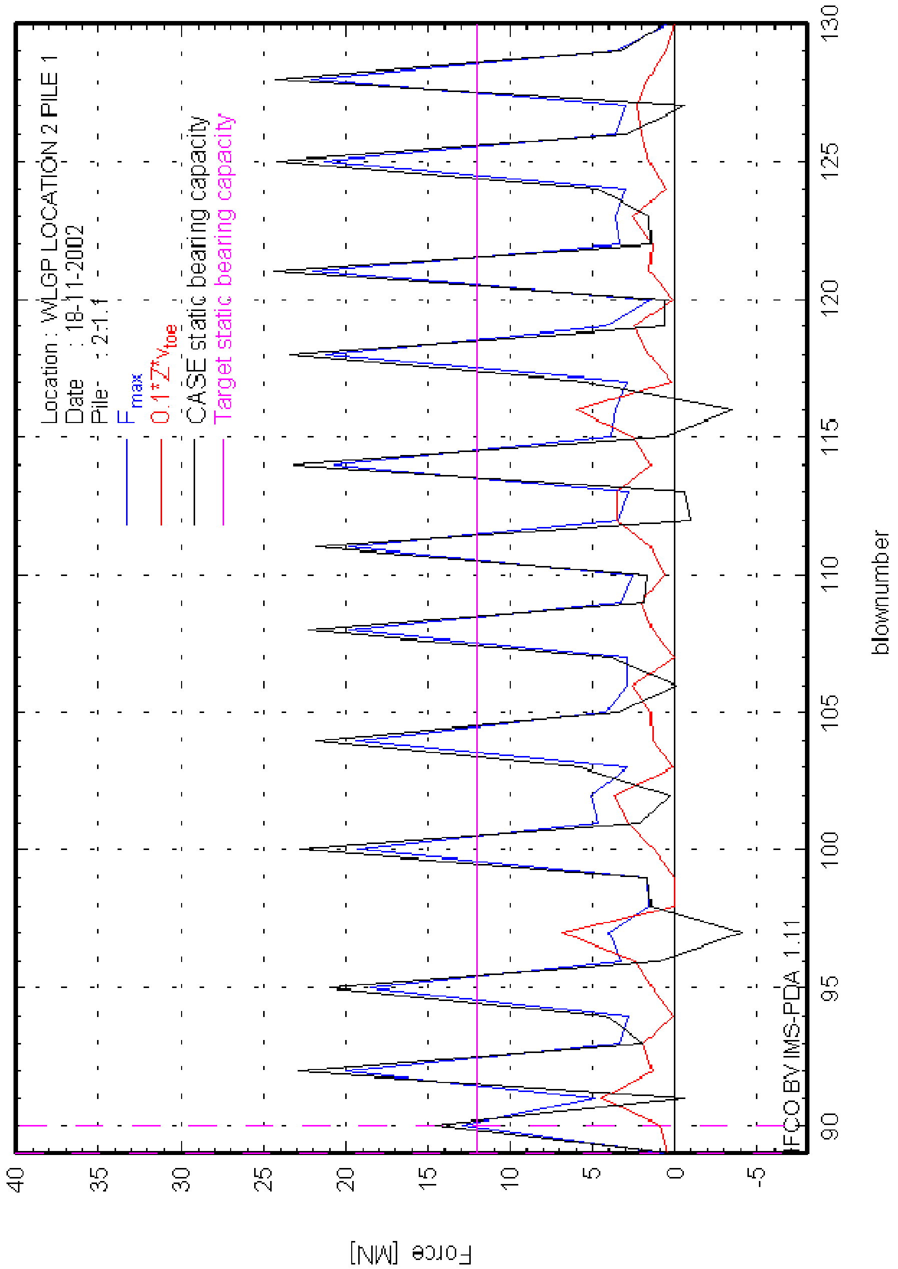
TEST RESULTS:

BLOW NO.		125				
FINAL SET		-	[mm / BLOW]			
NEAREST BOREHOLE RECORD		-				
DATE OF LAST HAMMERING		29/10/2002				
CASE STATIC BEARING CAPACITY WITH VARIOUS DAMPING FACTORS	J_c = 0.0	-	[kN]	J_c = 0.5	-	[kN]
	J_c = 0.1	24100	[kN]	J_c = 0.6	-	[kN]
	J_c = 0.2	-	[kN]	J_c = 0.7	-	[kN]
	J_c = 0.3	-	[kN]	J_c = 0.8	-	[kN]
	J_c = 0.4	-	[kN]	J_c = 0.9	-	[kN]
MAX. FORCE IN SENSOR LEVEL		74.905	[kN]			
MAX. COMPRESSIVE STRESS		211	[MPa]			
MAX. TENSILE STRESS		-	[MPa]			
MAX. ENERGY TRANSFERRED		174	[kNm]			
HAMMER DROP HEIGHT		-	[m]			
HAMMER RATED ENERGY		200	[kNm]			
ENERGY TRANSFERRED		87%	[%]			
PILE INTEGRITY		100	[%]			

PILE DAMAGED:

PILE DAMAGED AT LEVEL - [m] FROM THE LEVEL OF GAUGES - [m]

FIGURE 6 - 'CASE' STATIC BEARING CAPACITY FOR ALL BLOWS OF THE RESTRIKE TEST



TEST RESULT - DYNAMIC PILE TEST (TP3-2)

CLIENT: BESIX S.A. – LIBYA BRANCH

PROJECT: WEST LIBYA GAS PROJECT – JETTY WORKS

DATE: 18/11/2002 *TYPE OF TEST:* ☑ RESTRIKE ☐ END OF DRIVE

EQUIPMENT:

PDA	-	ANGLE METER	-	MEASURING TAPE	-
		[F1]		[F2]	
STRAIN GAUGE	EQU. NO.	-	EQU. NO.	-	
	CAL	-	CAL	-	
		[A1]		[A2]	
ACCELEROMETER	EQU. NO.	-	EQU. NO.	-	
	CAL	-	CAL	-	

HAMMER PROPERTIES:

HAMMER TYPE	☑ HYDRAULIC	☐ DROP HAMMER	☐ SPECIFIED	
TYPE OF CUSHION	☐ PLYWOOD	☐ RUBBER	☑ SPECIFIED	NONE
SERIAL NO.	-	WEIGHT OF RAM	- [ton]	

PILE PROPERTIES:

PILE NO.	TP3-2	L_t	41 [m]	ELASTIC MODULUS	210000 [MPa]
TYPE	Ø1422 STEEL PIPE	L_e	36.75 [m]	YIELD STRENGTH	355 [MPa]
INCLINATION	- []	L_p	16.40 [m]	REQUIRE CAPACITY	12000 [kN]
AREA	0.101 [m²]	GRADE	X52	WAVE SPEED	5172 [m/s]
COATING	NONE	IMPEDANCE	4.06E+03 [kNs/m]	SPECIFIC WEIGHT	7.85 [kN/m³]

TEST RESULTS:

BLOW NO.		3683				
FINAL SET		-	[mm / BLOW]			
NEAREST BOREHOLE RECORD		-				
DATE OF LAST HAMMERING		18/11/2002				
CASE STATIC BEARING CAPACITY WITH VARIOUS DAMPING FACTORS	$J_c = 0.0$	-	[kN]	$J_c = 0.5$	-	[kN]
	$J_c = 0.1$	26400	[kN]	$J_c = 0.6$	-	[kN]
	$J_c = 0.2$	-	[kN]	$J_c = 0.7$	-	[kN]
	$J_c = 0.3$	-	[kN]	$J_c = 0.8$	-	[kN]
	$J_c = 0.4$	-	[kN]	$J_c = 0.9$	-	[kN]
MAX. FORCE IN SENSOR LEVEL		7881	[kN]			
MAX. COMPRESSIVE STRESS		222	[MPa]			
MAX. TENSILE STRESS		-	[MPa]			
MAX. ENERGY TRANSFERRED		189	[kNm]			
HAMMER DROP HEIGHT		-	[m]			
HAMMER RATED ENERGY		200	[kNm]			
ENERGY TRANSFERRED		95%	[%]			
PILE INTEGRITY		100	[%]			

PILE DAMAGED:

PILE DAMAGED AT LEVEL - [m] FROM THE LEVEL OF GAUGES - [m]

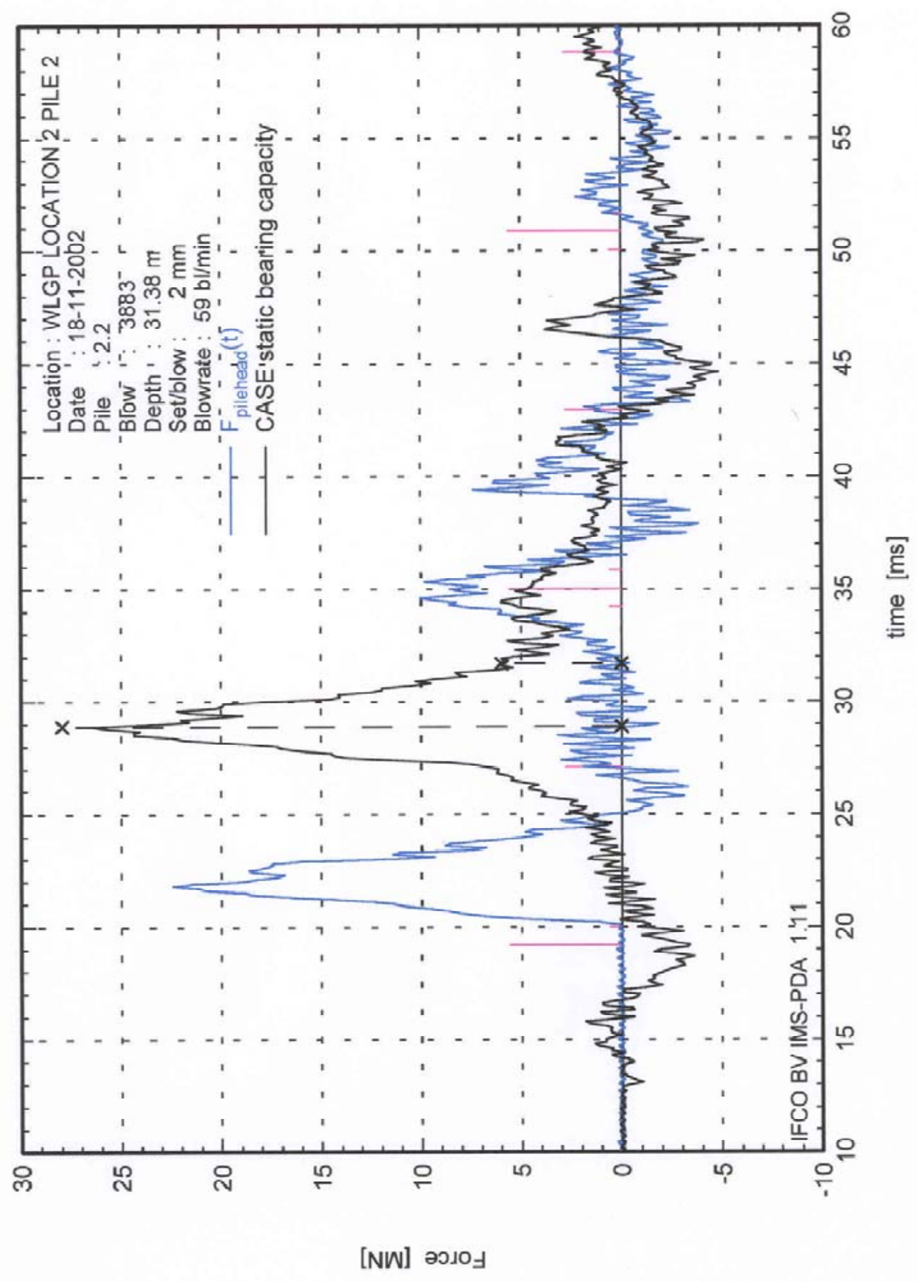


FIGURE 6 - 'CASE' STATIC BEARING CAPACITY VS BLOW

TEST RESULT - DYNAMIC PILE TEST (TP5-2)

CLIENT:	BESIX S.A. – LIBYA BRANCH
PROJECT:	WEST LIBYA GAS PROJECT – JETTY WORKS
DATE:	18/11/2002
TYPE OF TEST:	☑ RESTRIKE ☐ END OF DRIVE

EQUIPMENT:

PDA	-	ANGLE METER	-	MEASURING TAPE	-

		[F1]		[F2]
STRAIN GAUGE	EQU. NO.	-	EQU. NO.	-
	CAL	-	CAL	-
		[A1]		[A2]
ACCELEROMETER	EQU. NO.	-	EQU. NO.	-
	CAL	-	CAL	-

HAMMER PROPERTIES:

HAMMER TYPE	☑ HYDRAULIC	☐ DROP HAMMER	☐ SPECIFIED	
TYPE OF CUSHION	☐ PLYWOOD	☐ RUBBER	☑ SPECIFIED	NONE
SERIAL NO.	-	WEIGHT OF RAM	- [ton]	

PILE PROPERTIES:

PILE NO.	TP5-2	L_t	41 [m]	ELASTIC MODULUS	210000 [MPa]
TYPE	Ø1422 STEEL PIPE	L_e	36.75 [m]	YIELD STRENGTH	355 [MPa]
INCLINATION	- []	L_p	17.55 [m]	REQUIRE CAPACITY	12000 [kN]
AREA	0.101 [m²]	GRADE	X52	WAVE SPEED	5172 [m/s]
COATING	NONE	IMPEDANCE	4.06E+03 [kNs/m]	SPECIFIC WEIGHT	7.85 [kN/m³]

TEST RESULTS:

BLOW NO.	122		
FINAL SET	-	[mm / BLOW]	
NEAREST BOREHOLE RECORD	-		
DATE OF LAST HAMMERING	18/11/2002		

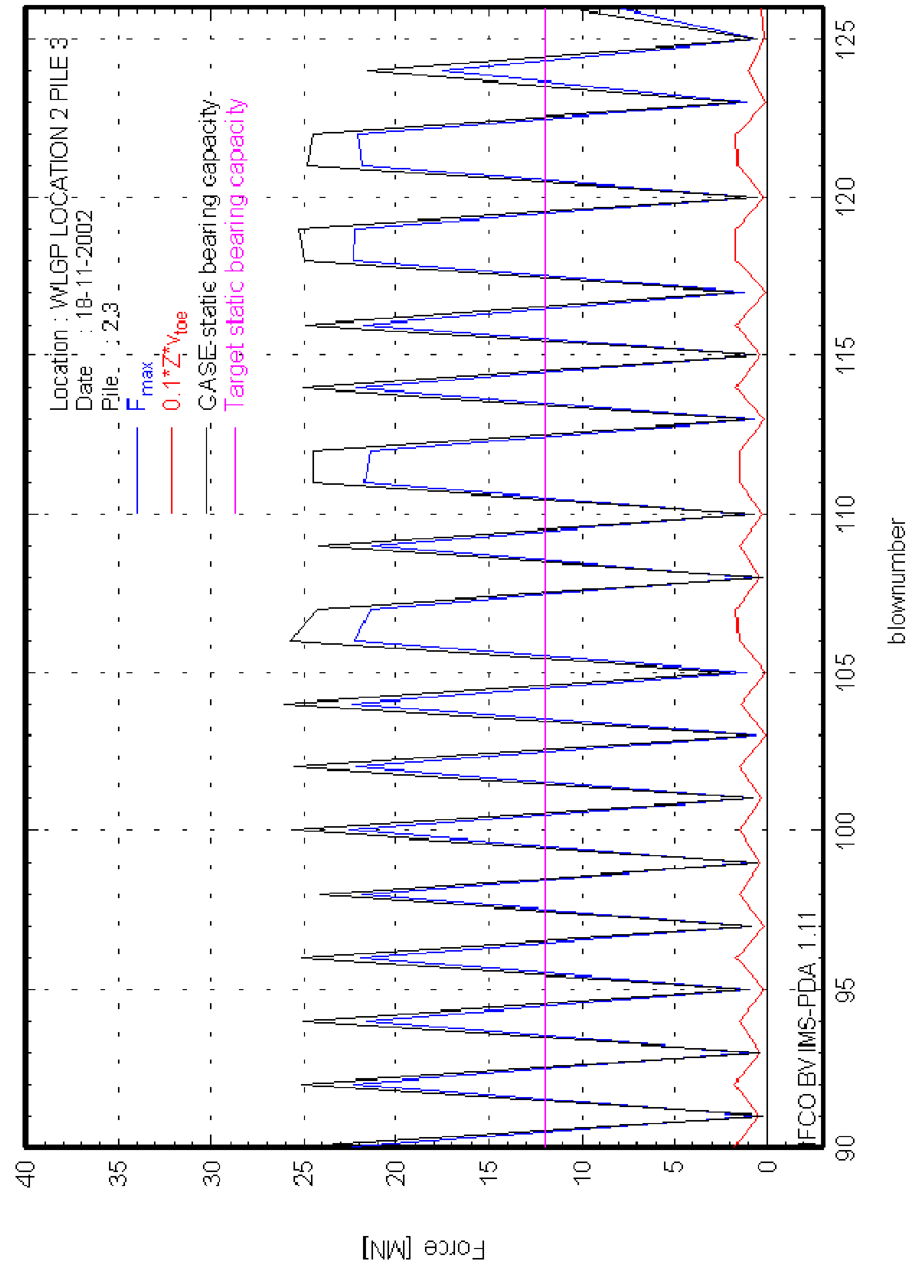
CASE STATIC BEARING CAPACITY WITH VARIOUS DAMPING FACTORS	J_c = 0.0	- [kN]	J_c = 0.5	- [kN]		
	J_c = 0.1	24500 [kN]	J_c = 0.6	- [kN]		
	J_c = 0.2	- [kN]	J_c = 0.7	- [kN]		
	J_c = 0.3	- [kN]	J_c = 0.8	- [kN]		
	J_c = 0.4	- [kN]	J_c = 0.9	- [kN]		

MAX. FORCE IN SENSOR LEVEL	77.39	[kN]
MAX. COMPRESSIVE STRESS	218	[MPa]
MAX. TENSILE STRESS	-	[MPa]
MAX. ENERGY TRANSFERRED	195	[kNm]
HAMMER DROP HEIGHT	-	[m]
HAMMER RATED ENERGY	200	[kNm]
ENERGY TRANSFERRED	98%	[%]
PILE INTEGRITY	100	[%]

PILE DAMAGED:

PILE DAMAGED AT LEVEL	- [m]	FROM THE LEVEL OF GAUGES	- [m]

FIGURE 6 - 'CASE' STATIC BEARING CAPACITY FOR EACH BLOW OF THE RESTRIKE TEST



TEST RESULT - DYNAMIC PILE TEST (TP3-1)

CLIENT:	BESIX S.A. – LIBYA BRANCH		
PROJECT:	WEST LIBYA GAS PROJECT – JETTY WORKS		
DATE:	12/11/2002	*TYPE OF TEST:*	☑ RESTRIKE ☐ END OF DRIVE

EQUIPMENT:

PDA	-	ANGLE METER	-	MEASURING TAPE	-

		[F1]		[F2]
STRAIN GAUGE	EQU. NO.	-	EQU. NO.	-
	CAL	-	CAL	-
		[A1]		[A2]
ACCELEROMETER	EQU. NO.	-	EQU. NO.	-
	CAL	-	CAL	-

HAMMER PROPERTIES:

HAMMER TYPE	☑ HYDRAULIC	☐ DROP HAMMER	☐ SPECIFIED	
TYPE OF CUSHION	☐ PLYWOOD	☐ RUBBER	☑ SPECIFIED	NONE
SERIAL NO.	-	WEIGHT OF RAM	- [ton]	

PILE PROPERTIES:

PILE NO.	TP3-1	L_t	41 [m]	ELASTIC MODULUS	210000 [MPa]
TYPE	Ø1422 STEEL PIPE	L_e	36.75 [m]	YIELD STRENGTH	355 [MPa]
INCLINATION	- [°]	L_p	24.00 [m]	REQUIRE CAPACITY	12000 [kN]
AREA	0.101 [m²]	GRADE	X52	WAVE SPEED	5172 [m/s]
COATING	NONE	IMPEDANCE	4.06E+03 [kNs/m]	SPECIFIC WEIGHT	7.85 [kN/m³]

TEST RESULTS:

BLOW NO.		2270	
FINAL SET		-	[mm / BLOW]
NEAREST BOREHOLE RECORD		-	
DATE OF LAST HAMMERING		12/11/2002	

CASE STATIC BEARING CAPACITY WITH VARIOUS DAMPING FACTORS	$J_c = 0.0$	- [kN]	$J_c = 0.5$	- [kN]
	$J_c = 0.1$	13900 [kN]	$J_c = 0.6$	- [kN]
	$J_c = 0.2$	- [kN]	$J_c = 0.7$	- [kN]
	$J_c = 0.3$	- [kN]	$J_c = 0.8$	- [kN]
	$J_c = 0.4$	- [kN]	$J_c = 0.9$	- [kN]

MAX. FORCE IN SENSOR LEVEL	67.095	[kN]
MAX. COMPRESSIVE STRESS	189	[MPa]
MAX. TENSILE STRESS	-	[MPa]
MAX. ENERGY TRANSFERRED	119	[kNm]
HAMMER DROP HEIGHT	-	[m]
HAMMER RATED ENERGY	150	[kNm]
ENERGY TRANSFERRED	79%	[%]
PILE INTEGRITY	100	[%]

PILE DAMAGED:

PILE DAMAGED AT LEVEL	- [m]	FROM THE LEVEL OF GAUGES	- [m]

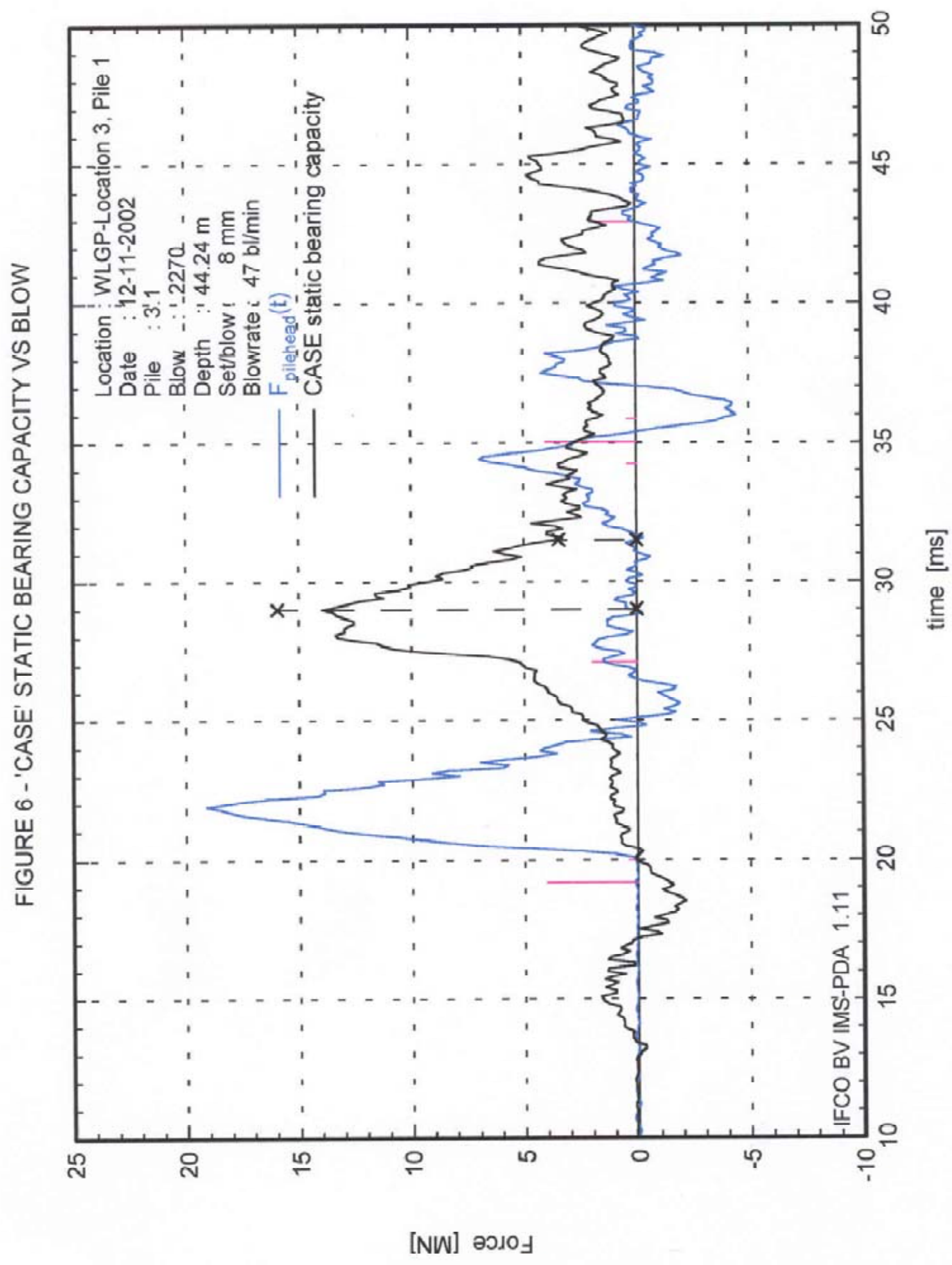


FIGURE 6 - 'CASE' STATIC BEARING CAPACITY VS BLOW
Location : WLGP-Location 3, Pile 1
Date : 12-11-2002
Pile : 3'1
Blow : 2270
Depth : 44.24 m
Set/blow : 8 mm
Blowrate : 47 bl/min
$F_{pilehead}(t)$
CASE static bearing capacity
IFCO BV IMS-PDA 1.11
Force [MN]
time [ms]
25
20
15
10
5
0
-5
-10
10
15
20
25
30
35
40
45
50

TEST RESULT - DYNAMIC PILE TEST (TP4-1)

CLIENT: BESIX S.A. – LIBYA BRANCH

PROJECT: WEST LIBYA GAS PROJECT – JETTY WORKS

DATE: 13/11/2002

TYPE OF TEST: ☑ RESTRIKE ☐ END OF DRIVE

EQUIPMENT:

PDA	-	ANGLE METER	-	MEASURING TAPE	-
		[F1]		[F2]	
STRAIN GAUGE	EQU. NO.	-	EQU. NO.	-	
	CAL	-	CAL	-	
		[A1]		[A2]	
ACCELEROMETER	EQU. NO.	-	EQU. NO.	-	
	CAL	-	CAL	-	

HAMMER PROPERTIES:

HAMMER TYPE	☑ HYDRAULIC	☐ DROP HAMMER	☐ SPECIFIED	
TYPE OF CUSHION	☐ PLYWOOD	☐ RUBBER	☑ SPECIFIED	NONE
SERIAL NO.	-	WEIGHT OF RAM	- [ton]	

PILE PROPERTIES:

PILE NO.	TP4-1	L_t	48 [m]	ELASTIC MODULUS	210000 [MPa]
TYPE	Ø1422 STEEL PIPE	L_e	43.75 [m]	YIELD STRENGTH	355 [MPa]
INCLINATION	- []	L_p	24.00 [m]	REQUIRE CAPACITY	12000 [kN]
AREA	0.101 [m²]	GRADE	X52	WAVESPEED	5172 [m/s]
COATING	NONE	IMPEDANCE	4.06E+03 [kNs/m]	SPECIFIC WEIGHT	7.85 [kN/m³]

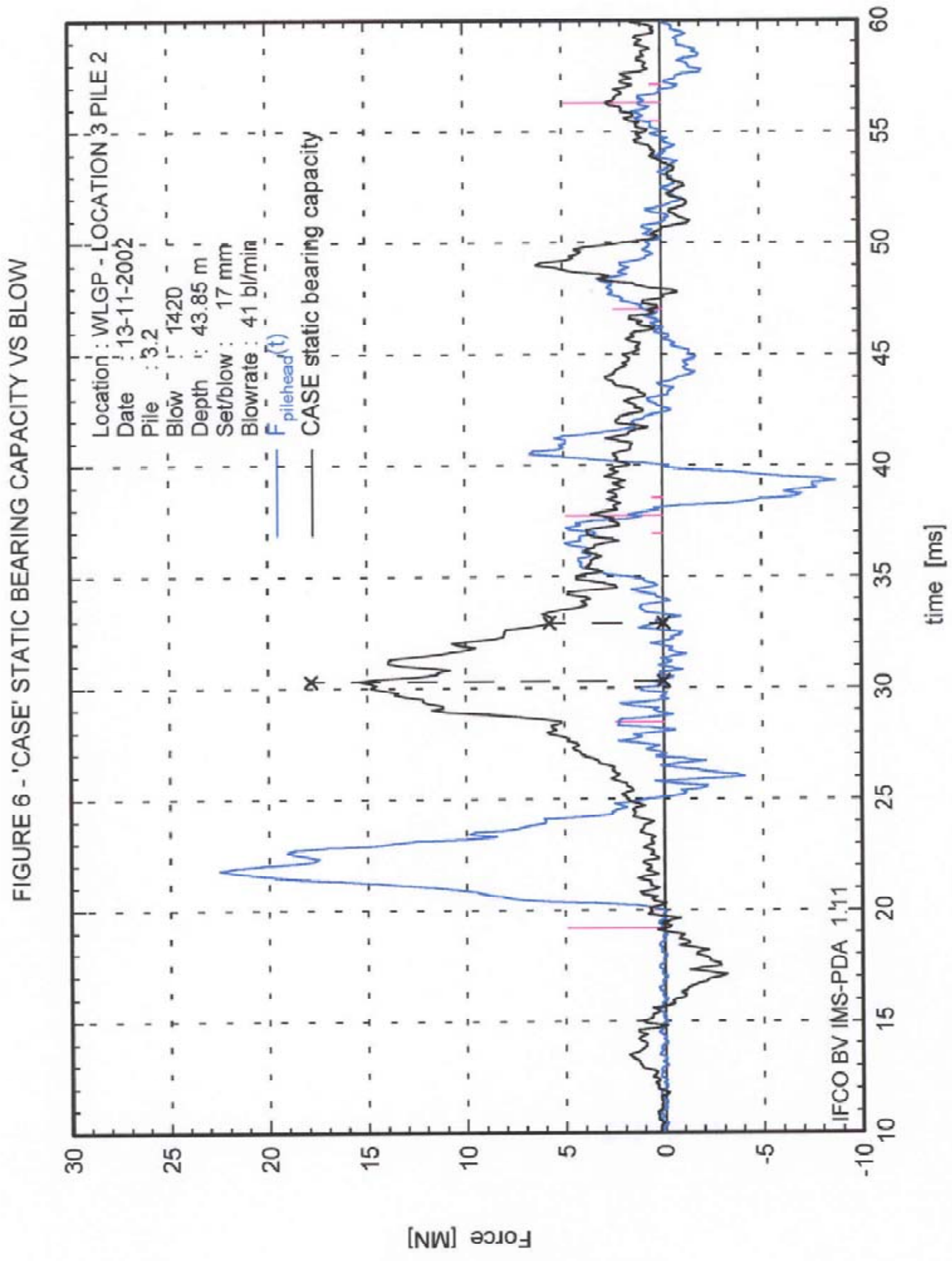
TEST RESULTS:

BLOW NO.		1420			
FINAL SET		- [mm / BLOW]			
NEAREST BOREHOLE RECORD		-			
DATE OF LAST HAMMERING		13/11/2002			
CASE STATIC BEARING CAPACITY WITH VARIOUS DAMPING FACTORS	J_c = 0.0	- [kN]	J_c = 0.5	- [kN]	
	J_c = 0.1	15400 [kN]	J_c = 0.6	- [kN]	
	J_c = 0.2	- [kN]	J_c = 0.7	- [kN]	
	J_c = 0.3	- [kN]	J_c = 0.8	- [kN]	
	J_c = 0.4	- [kN]	J_c = 0.9	- [kN]	
MAX. FORCE IN SENSOR LEVEL		71.71 [kN]			
MAX. COMPRESSIVE STRESS		202 [MPa]			
MAX. TENSILE STRESS		- [MPa]			
MAX. ENERGY TRANSFERRED		176 [kNm]			
HAMMER DROP HEIGHT		- [m]			
HAMMER RATED ENERGY		200 [kNm]			
ENERGY TRANSFERRED		88% [%]			
PILE INTEGRITY		100 [%]			

PILE DAMAGED:

PILE DAMAGED AT LEVEL - [m] FROM THE LEVEL OF GAUGES - [m]

FIGURE 6 - 'CASE' STATIC BEARING CAPACITY VS BLOW



TEST RESULT - DYNAMIC PILE TEST (TP5-1)

CLIENT: BESIX S.A. – LIBYA BRANCH

PROJECT: WEST LIBYA GAS PROJECT – JETTY WORKS

DATE: 29/12/2002

TYPE OF TEST: ☑ RESTRIKE ☐ END OF DRIVE

EQUIPMENT:

PDA	-	ANGLE METER	-	MEASURING TAPE	-
		[F1]		[F2]	
STRAIN GAUGE	EQU. NO.	-	EQU. NO.	-	
	CAL	-	CAL	-	
		[A1]		[A2]	
ACCELEROMETER	EQU. NO.	-	EQU. NO.	-	
	CAL	-	CAL	-	

HAMMER PROPERTIES:

HAMMER TYPE	☑ HYDRAULIC	☐ DROP HAMMER	☐ SPECIFIED	
TYPE OF CUSHION	☐ PLYWOOD	☐ RUBBER	☑ SPECIFIED	NONE
SERIAL NO.	-	WEIGHT OF RAM	- [ton]	

PILE PROPERTIES:

PILE NO.	TP5-1	L_t	41 [m]	ELASTIC MODULUS	210000 [MPa]
TYPE	Ø1422 STEEL PIPE	L_e	38.50 [m]	YIELD STRENGTH	355 [MPa]
INCLINATION	- []	L_p	24.00 [m]	REQUIRE CAPACITY	12000 [kN]
AREA	0.101 [m²]	GRADE	X52	WAVE SPEED	5172 [m/s]
COATING	NONE	IMPEDANCE	4.06E+03 [kNs/m]	SPECIFIC WEIGHT	7.85 [kN/m³]

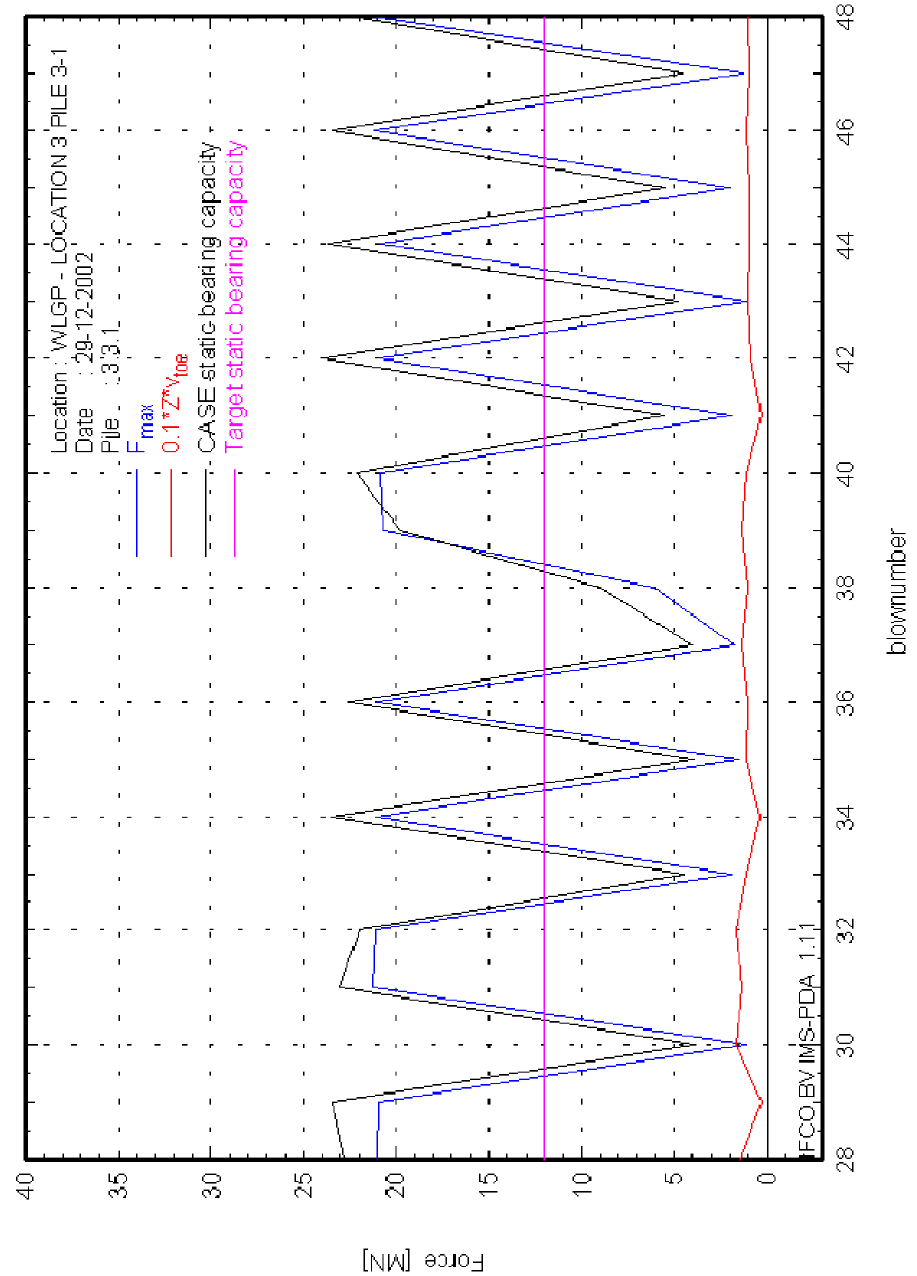
TEST RESULTS:

BLOW NO.		32				
FINAL SET		-	[mm / BLOW]			
NEAREST BOREHOLE RECORD		-				
DATE OF LAST HAMMERING		14/11/2002				
CASE STATIC BEARING CAPACITY WITH VARIOUS DAMPING FACTORS	J_c = 0.0	-	[kN]	J_c = 0.5	-	[kN]
	J_c = 0.1	22000	[kN]	J_c = 0.6	-	[kN]
	J_c = 0.2	-	[kN]	J_c = 0.7	-	[kN]
	J_c = 0.3	-	[kN]	J_c = 0.8	-	[kN]
	J_c = 0.4	-	[kN]	J_c = 0.9	-	[kN]
MAX. FORCE IN SENSOR LEVEL		74.0175	[kN]			
MAX. COMPRESSIVE STRESS		208.5	[MPa]			
MAX. TENSILE STRESS		-	[MPa]			
MAX. ENERGY TRANSFERRED		165.7	[kNm]			
HAMMER DROP HEIGHT		-	[m]			
HAMMER RATED ENERGY		200	[kNm]			
ENERGY TRANSFERRED		83%	[%]			
PILE INTEGRITY		100	[%]			

PILE DAMAGED:

PILE DAMAGED AT LEVEL - [m] FROM THE LEVEL OF GAUGES - [m]

FIGURE 6 - 'CASE' STATIC BEARING CAPACITY FOR THE RESTRIKE TEST



TEST RESULT - DYNAMIC PILE TEST (TP4-2)

CLIENT: BESIX S.A. – LIBYA BRANCH

PROJECT: WEST LIBYA GAS PROJECT – JETTY WORKS

DATE: 15/11/2002 *TYPE OF TEST:* ☑ RESTRIKE ☐ END OF DRIVE

EQUIPMENT:

PDA	-	ANGLE METER	-	MEASURING TAPE	-

		[F1]		[F2]
STRAIN GAUGE	EQU. NO.	-	EQU. NO.	-
	CAL	-	CAL	-
		[A1]		[A2]
ACCELEROMETER	EQU. NO.	-	EQU. NO.	-
	CAL	-	CAL	-

HAMMER PROPERTIES:

HAMMER TYPE ☑ HYDRAULIC ☐ DROP HAMMER ☐ SPECIFIED

TYPE OF CUSHION ☐ PLYWOOD ☐ RUBBER ☑ SPECIFIED NONE

SERIAL NO. - WEIGHT OF RAM - [ton]

PILE PROPERTIES:

PILE NO.	TP4-2	L_t	48 [m]	ELASTIC MODULUS	210000 [MPa]
TYPE	Ø1422 STEEL PIPE	L_e	43.75 [m]	YIELD STRENGTH	355 [MPa]
INCLINATION	- []	L_p	24.00 [m]	REQUIRE CAPACITY	12000 [kN]
AREA	0.101 [m²]	GRADE	X52	WAVE SPEED	5172 [m/s]
COATING	NONE	IMPEDANCE	4.06E+03 [kNs/m]	SPECIFIC WEIGHT	7.85 [kN/m³]

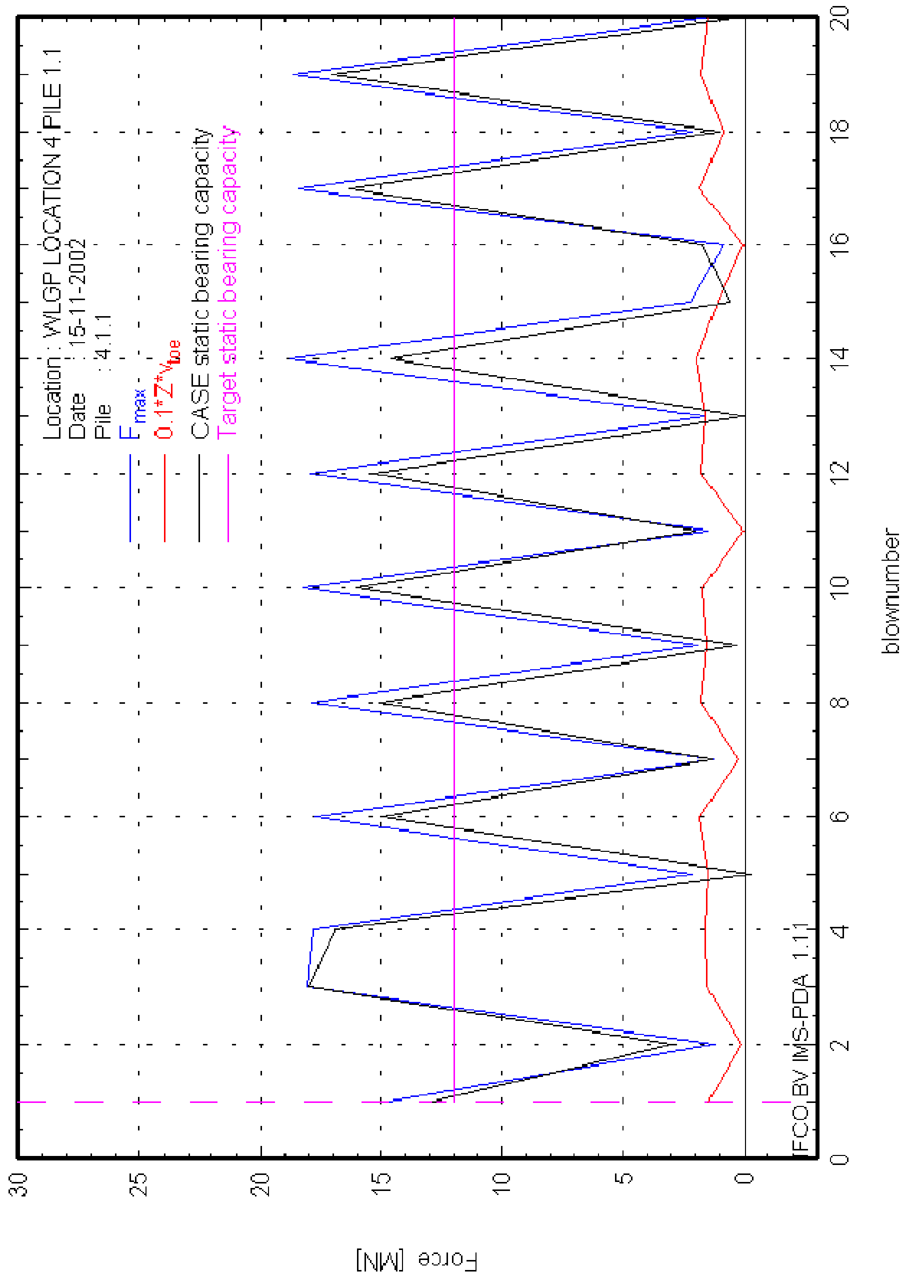
TEST RESULTS:

BLOW NO.		3			
FINAL SET		- [mm / BLOW]			
NEAREST BOREHOLE RECORD		-			
DATE OF LAST HAMMERING		15/11/2002			
CASE STATIC BEARING CAPACITY WITH VARIOUS DAMPING FACTORS	J_c = 0.0	- [kN]	J_c = 0.5	- [kN]
	J_c = 0.1	18000 [kN]	J_c = 0.6	- [kN]
	J_c = 0.2	- [kN]	J_c = 0.7	- [kN]
	J_c = 0.3	- [kN]	J_c = 0.8	- [kN]
	J_c = 0.4	- [kN]	J_c = 0.9	- [kN]
MAX. FORCE IN SENSOR LEVEL		63.545 [kN]		
MAX. COMPRESSIVE STRESS		17.9 [MPa]		
MAX. TENSILE STRESS		- [MPa]		
MAX. ENERGY TRANSFERRED		114 [kNm]		
HAMMER DROP HEIGHT		- [m]		
HAMMER RATED ENERGY		150 [kNm]		
ENERGY TRANSFERRED		76% [%]		
PILE INTEGRITY		100 [%]		

PILE DAMAGED:

PILE DAMAGED AT LEVEL - [m] FROM THE LEVEL OF GAUGES - [m]

FIGURE 6 - 'CASE' STATIC BEARING CAPACITY FOR ALL BLOWS OF THE RESTRIKE TEST



TEST RESULT - DYNAMIC PILE TEST (TP4-3)

CLIENT: BESIX S.A. – LIBYA BRANCH

PROJECT: WEST LIBYA GAS PROJECT – JETTY WORKS

DATE: 16/11/2002 *TYPE OF TEST:* ☑ RESTRIKE ☐ END OF DRIVE

EQUIPMENT:

PDA	-	ANGLE METER	-	MEASURING TAPE	-
			[F1]		[F2]
STRAIN GAUGE	EQU. NO.		-	EQU. NO.	-
	CAL		-	CAL	-
			[A1]		[A2]
ACCELEROMETER	EQU. NO.		-	EQU. NO.	-
	CAL		-	CAL	-

HAMMER PROPERTIES:

HAMMER TYPE	☑ HYDRAULIC	☐ DROP HAMMER	☐ SPECIFIED	
TYPE OF CUSHION	☐ PLYWOOD	☐ RUBBER	☑ SPECIFIED	NONE
SERIAL NO.	-	WEIGHT OF RAM	- [ton]	

PILE PROPERTIES:

PILE NO.	TP4-3	L_t	48 [m]	ELASTIC MODULUS	210000 [MPa]
TYPE	Ø1422 STEEL PIPE	L_e	43.75 [m]	YIELD STRENGTH	355 [MPa]
INCLINATION	- []	L_p	24.00 [m]	REQUIRE CAPACITY	12000 [kN]
AREA	0.101 [m²]	GRADE	X52	WAVE SPEED	5172 [m/s]
COATING	NONE	IMPEDANCE	4.06E+03 [kNs/m]	SPECIFIC WEIGHT	7.85 [kN/m³]

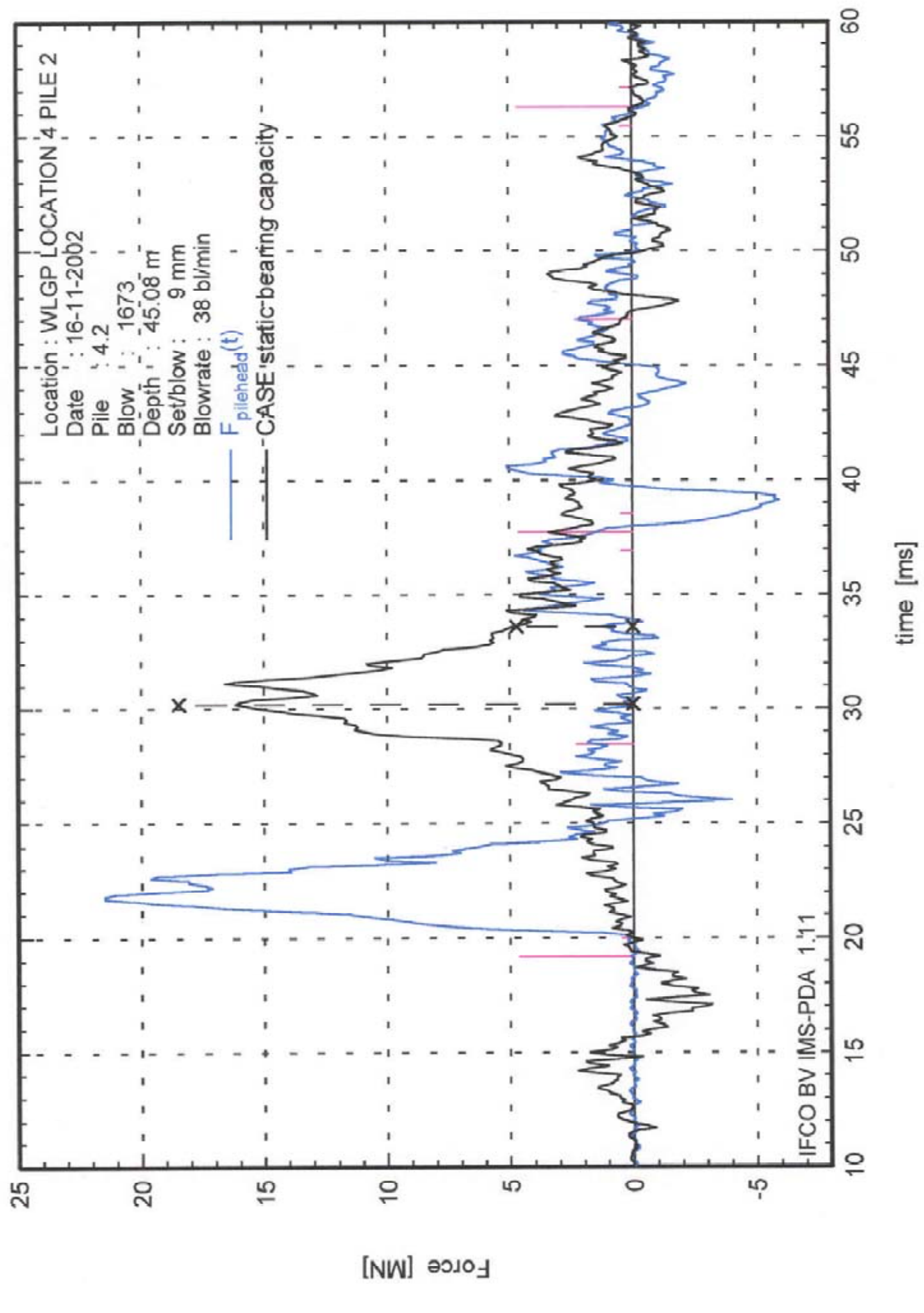
TEST RESULTS:

BLOW NO.		1673				
FINAL SET		-	[mm / BLOW]			
NEAREST BOREHOLE RECORD		-				
DATE OF LAST HAMMERING		16/11/2002				
CASE STATIC BEARING CAPACITY WITH VARIOUS DAMPING FACTORS	$J_c = 0.0$	-	[kN]	$J_c = 0.5$	-	[kN]
	$J_c = 0.1$	16100	[kN]	$J_c = 0.6$	-	[kN]
	$J_c = 0.2$	-	[kN]	$J_c = 0.7$	-	[kN]
	$J_c = 0.3$	-	[kN]	$J_c = 0.8$	-	[kN]
	$J_c = 0.4$	-	[kN]	$J_c = 0.9$	-	[kN]
MAX. FORCE IN SENSOR LEVEL		75.615	[kN]			
MAX. COMPRESSIVE STRESS		213	[MPa]			
MAX. TENSILE STRESS		-	[MPa]			
MAX. ENERGY TRANSFERRED		180	[kNm]			
HAMMER DROP HEIGHT		-	[m]			
HAMMER RATED ENERGY		200	[kNm]			
ENERGY TRANSFERRED		90%	[%]			
PILE INTEGRITY		100	[%]			

PILE DAMAGED:

PILE DAMAGED AT LEVEL - [m] FROM THE LEVEL OF GAUGES - [m]

FIGURE 6 - 'CASE' STATIC BEARING CAPACITY VS BLOW



TEST RESULT - DYNAMIC PILE TEST (TP6-2)

CLIENT: BESIX S.A. – LIBYA BRANCH

PROJECT: WEST LIBYA GAS PROJECT – JETTY WORKS

DATE: 16/11/2002

TYPE OF TEST: ☑ RESTRIKE ☐ END OF DRIVE

EQUIPMENT:

PDA	-	ANGLE METER	-	MEASURING TAPE	-

		[F1]		[F2]
STRAIN GAUGE	EQU. NO.	-	EQU. NO.	-
	CAL	-	CAL	-
		[A1]		[A2]
ACCELEROMETER	EQU. NO.	-	EQU. NO.	-
	CAL	-	CAL	-

HAMMER PROPERTIES:

HAMMER TYPE	☑ HYDRAULIC	☐ DROP HAMMER	☐ SPECIFIED	
TYPE OF CUSHION	☐ PLYWOOD	☐ RUBBER	☑ SPECIFIED	NONE
SERIAL NO.	-	WEIGHT OF RAM	- [ton]	

PILE PROPERTIES:

PILE NO.	TP6-2	L_t	48 [m]	ELASTIC MODULUS	210000 [MPa]
TYPE	Ø1422 STEEL PIPE	L_e	43.75 [m]	YIELD STRENGTH	355 [MPa]
INCLINATION	- []	L_p	24.00 [m]	REQUIRE CAPACITY	12000 [kN]
AREA	0.101 [m²]	GRADE	X52	WAVE SPEED	5172 [m/s]
COATING	NONE	IMPEDANCE	4.06E+03 [kNs/m]	SPECIFIC WEIGHT	7.85 [kN/m³]

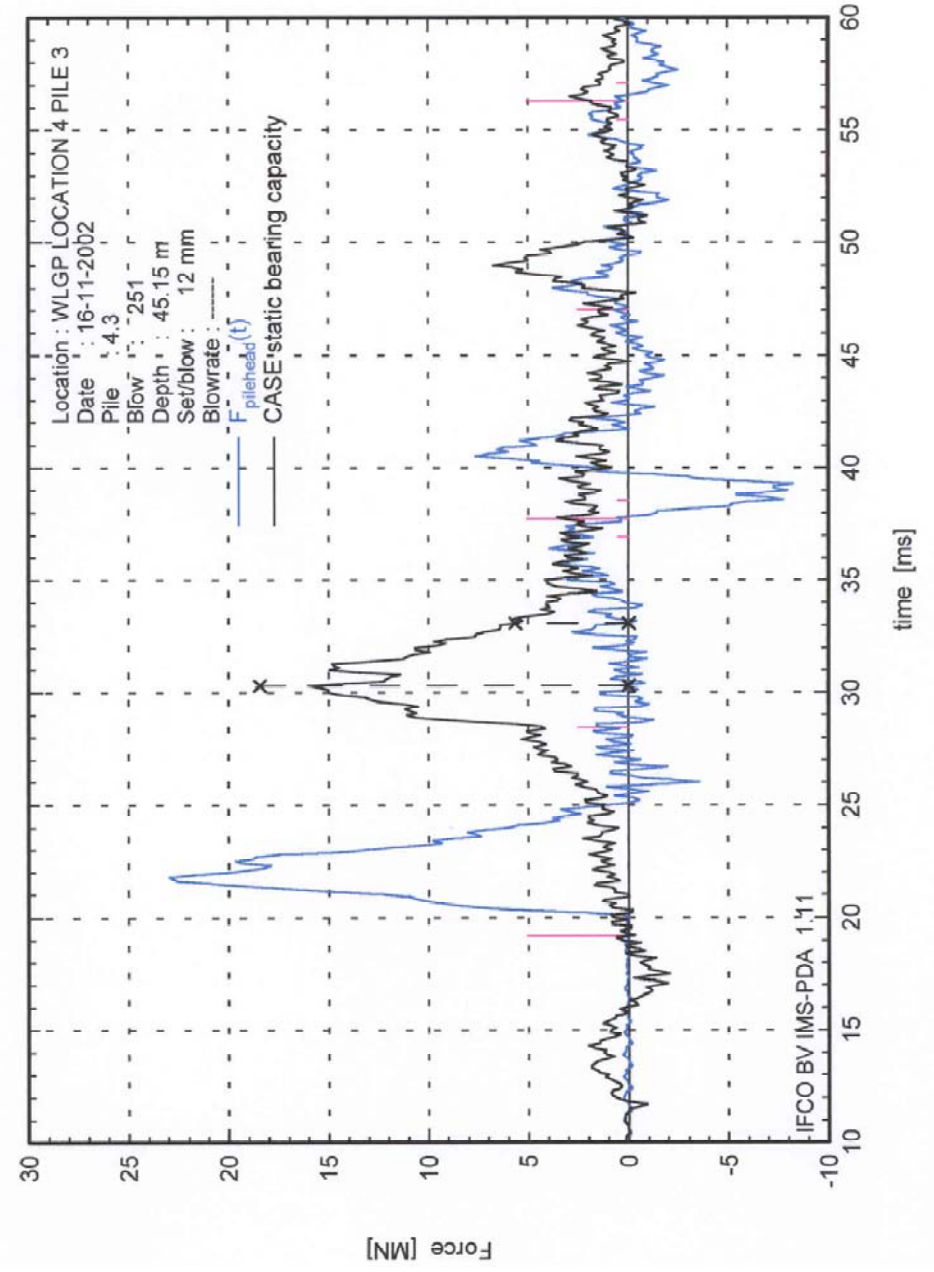
TEST RESULTS:

BLOW NO.		251			
FINAL SET		- [mm / BLOW]			
NEAREST BOREHOLE RECORD		-			
DATE OF LAST HAMMERING		16/11/2002			
CASE STATIC BEARING CAPACITY WITH VARIOUS DAMPING FACTORS	J_c = 0.0	- [kN]	J_c = 0.5	- [kN]
	J_c = 0.1	15900 [kN]	J_c = 0.6	- [kN]
	J_c = 0.2	- [kN]	J_c = 0.7	- [kN]
	J_c = 0.3	- [kN]	J_c = 0.8	- [kN]
	J_c = 0.4	- [kN]	J_c = 0.9	- [kN]
MAX. FORCE IN SENSOR LEVEL		80.585 [kN]			
MAX. COMPRESSIVE STRESS		227 [MPa]			
MAX. TENSILE STRESS		- [MPa]			
MAX. ENERGY TRANSFERRED		189 [kNm]			
HAMMER DROP HEIGHT		- [m]			
HAMMER RATED ENERGY		200 [kNm]			
ENERGY TRANSFERRED		95% [%]			
PILE INTEGRITY		100 [%]			

PILE DAMAGED:

PILE DAMAGED AT LEVEL - [m] FROM THE LEVEL OF GAUGES - [m]

FIGURE 6 - 'CASE' STATIC BEARING CAPACITY VS BLOW



TEST RESULT - DYNAMIC PILE TEST (EPC80-3)

CLIENT:	PROFICIENCY CONSTRUCTION (MACAU) CO., LTD.
PROJECT:	COTAI GALAXY HOTEL & CASINO
DATE:	25/1/2008
TYPE OF TEST:	☑ RESTRIKE ☐ END OF DRIVE

EQUIPMENT:

PDA	1423K	ANGLE METER	-	MEASURING TAPE	-
		[F1]		[F2]	
STRAIN GAUGE	EQU. NO.	5593	EQU. NO.	5594	
	CAL	-	CAL	-	
		[A1]		[A2]	
ACCELEROMETER	EQU. NO.	92275	EQU. NO.	92276	
	CAL	-	CAL	-	

HAMMER PROPERTIES:

HAMMER TYPE	☑ HYDRAULIC	☐ DROP HAMMER	☐ SPECIFIED	
TYPE OF CUSHION	☑ PLYWOOD	☐ RUBBER	☐ SPECIFIED	
SERIAL NO.	-	WEIGHT OF RAM	- [ton]	

PILE PROPERTIES:

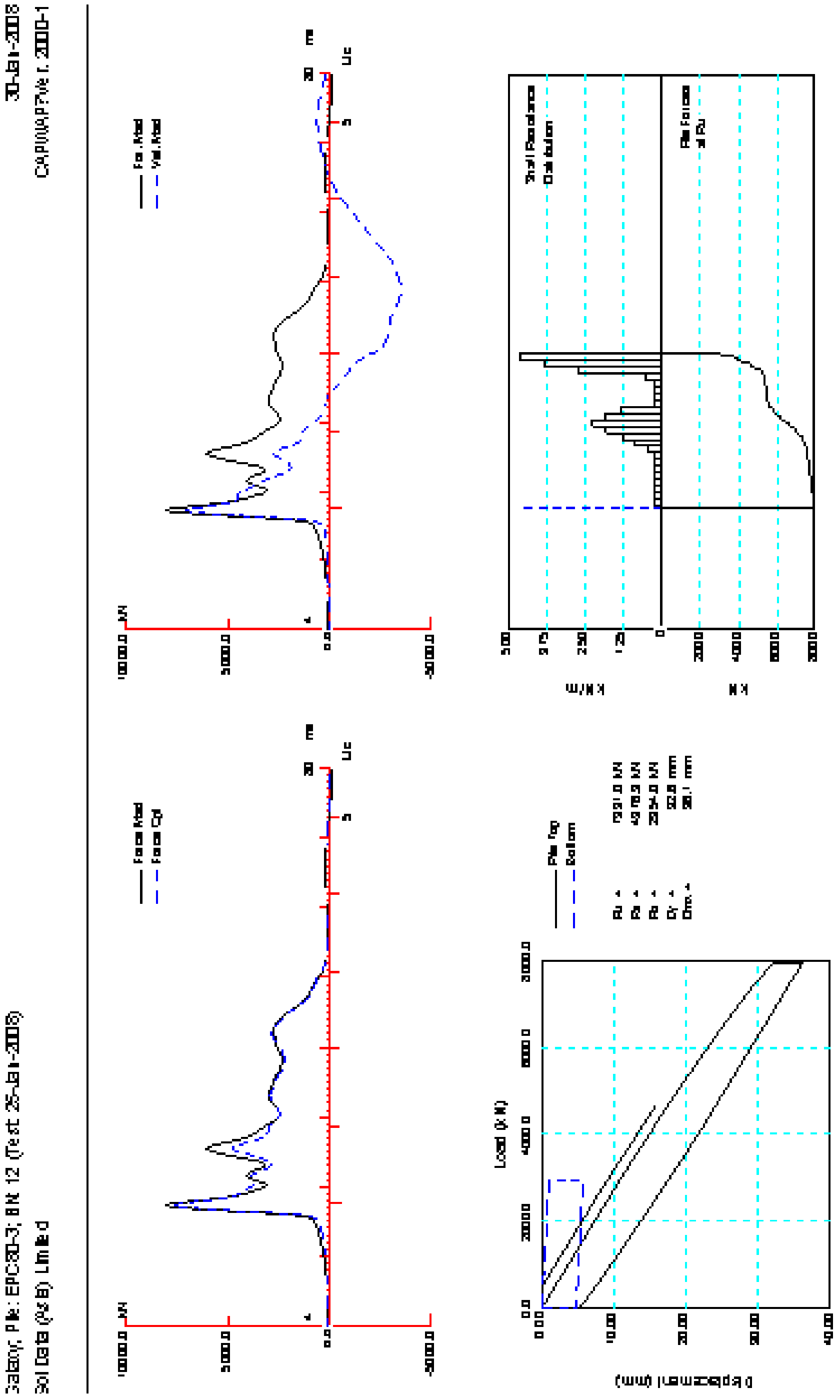
PILE NO.	EPC80-3	L_t	48 [m]	ELASTIC MODULUS	53688 [MPa]
TYPE	CONCRETE PILE	L_e	47.50 [m]	YIELD STRENGTH	80 [MPa]
INCLINATION	[]	L_p	47.00 [m]	REQUIRE CAPACITY	7000 [kN]
AREA	0.192 [m²]	GRADE	80	WAVESPEED	4500 [m/s]
COATING	NONE	IMPEDANCE	2.29E+03 [kNs/m]	SPECIFIC WEIGHT	26 [kN/m³]

TEST RESULTS:

BLOW NO.		12				
FINAL SET		-	[mm / BLOW]			
NEAREST BOREHOLE RECORD		-				
DATE OF LAST HAMMERING		-				
CASE STATIC BEARING CAPACITY WITH VARIOUS DAMPING FACTORS	J_c = 0.0	10369	[kN]	J_c = 0.5	7280	[kN]
	J_c = 0.1	9751	[kN]	J_c = 0.6	6662	[kN]
	J_c = 0.2	9133	[kN]	J_c = 0.7	6044	[kN]
	J_c = 0.3	8515	[kN]	J_c = 0.8	5651	[kN]
	J_c = 0.4	7897	[kN]	J_c = 0.9	5574	[kN]
MAX. FORCE IN SENSOR LEVEL		8952	[kN]			
MAX. COMPRESSIVE STRESS		46.6	[MPa]			
MAX. TENSILE STRESS		0	[MPa]			
MAX. ENERGY TRANSFERRED		81.2	[kNm]			
HAMMER DROP HEIGHT		-	[m]			
HAMMER RATED ENERGY		288	[kNm]			
ENERGY TRANSFERRED		28%	[%]			
PILE INTEGRITY		95	[%]			

PILE DAMAGED:

PILE DAMAGED AT LEVEL	- [m]	FROM THE LEVEL OF GAUGES	- [m]



TEST RESULT - DYNAMIC PILE TEST (EPC81A-3)

CLIENT: PROFICIENCY CONSTRUCTION (MACAU) CO., LTD.

PROJECT: COTAI GALAXY HOTEL & CASINO

DATE: 25/1/2008 *TYPE OF TEST:* ☑ RESTRIKE ☐ END OF DRIVE

EQUIPMENT:

PDA	1423K	ANGLE METER	-	MEASURING TAPE	-
		[F1]		[F2]	
STRAIN GAUGE	EQU. NO.	5593	EQU. NO.	5594	
	CAL	-	CAL	-	
		[A1]		[A2]	
ACCELEROMETER	EQU. NO.	92275	EQU. NO.	92276	
	CAL	-	CAL	-	

HAMMER PROPERTIES:

HAMMER TYPE	☑ HYDRAULIC	☐ DROP HAMMER	☐ SPECIFIED	
TYPE OF CUSHION	☑ PLYWOOD	☐ RUBBER	☐ SPECIFIED	
SERIAL NO.	-	WEIGHT OF RAM	- [ton]	

PILE PROPERTIES:

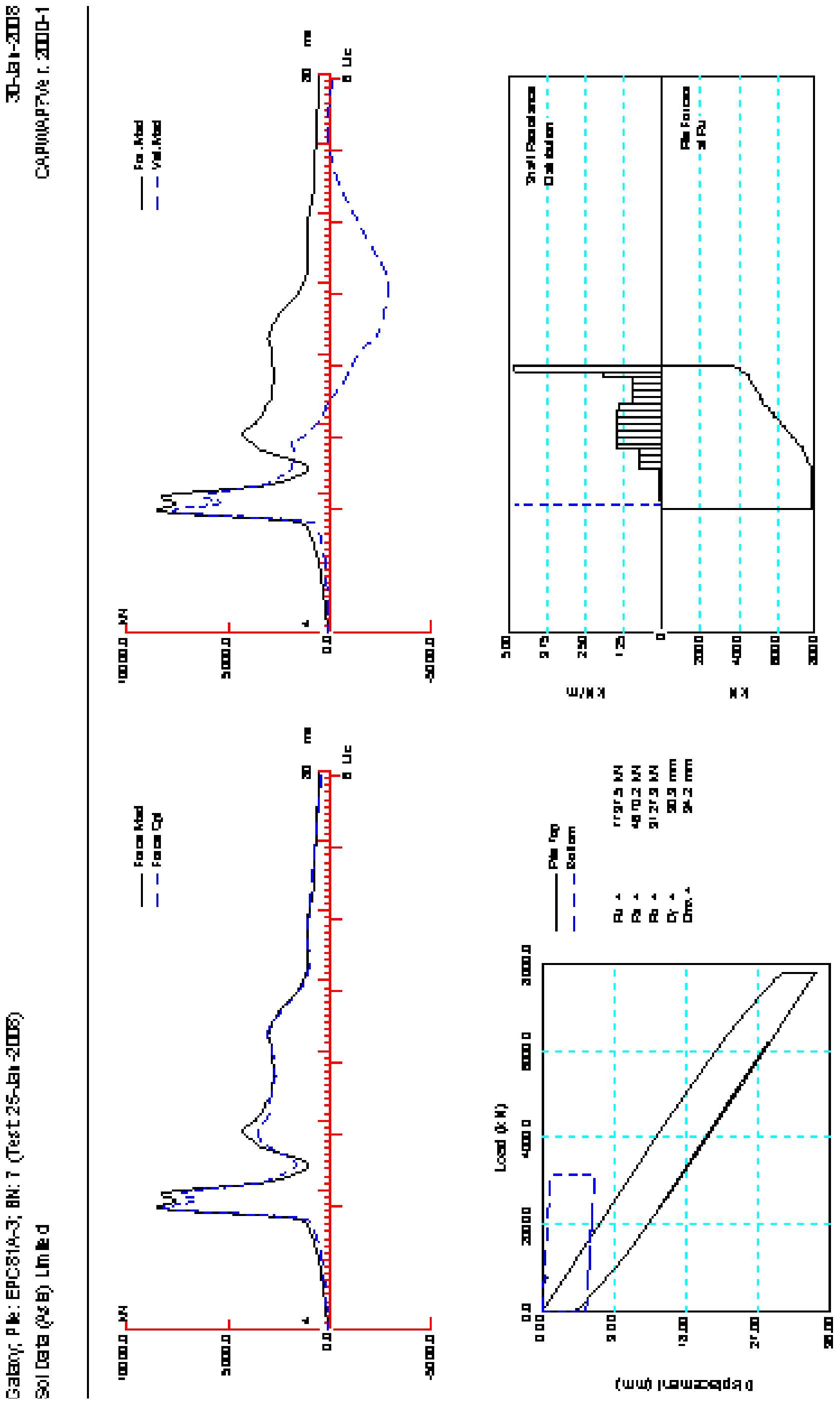
PILE NO.	EPC81A-3	L_i	48 [m]	ELASTIC MODULUS	53688 [MPa]
TYPE	CONCRETE PILE	L_E	44.00 [m]	YIELD STRENGTH	80 [MPa]
INCLINATION	0	L_P	42.50 [m]	REQUIRE CAPACITY	7000 [kN]
AREA	0.192 [m²]	GRADE	80	WAVE SPEED	4500 [m/s]
COATING	NONE	IMPEDANCE	2.29E+03 [kNs/m]	SPECIFIC WEIGHT	26 [kN/m³]

TEST RESULTS:

BLOW NO.		7				
FINAL SET		-	[mm / BLOW]			
NEAREST BOREHOLE RECORD		-				
DATE OF LAST HAMMERING		-				
CASE STATIC	J_c = 0.0	10406	[kN]	J_c = 0.5	7052	[kN]
BEARING	J_c = 0.1	9735	[kN]	J_c = 0.6	6382	[kN]
CAPACITY WITH VARIOUS	J_c = 0.2	9065	[kN]	J_c = 0.7	5711	[kN]
DAMPING	J_c = 0.3	8394	[kN]	J_c = 0.8	5106	[kN]
FACTORS	J_c = 0.4	7723	[kN]	J_c = 0.9	4841	[kN]
MAX. FORCE IN SENSOR LEVEL		9250	[kN]			
MAX. COMPRESSIVE STRESS		48.2	[MPa]			
MAX. TENSILE STRESS		0	[MPa]			
MAX. ENERGY TRANSFERRED		89.2	[kNm]			
HAMMER DROP HEIGHT		-	[m]			
HAMMER RATED ENERGY		288	[kNm]			
ENERGY TRANSFERRED		31%	[%]			
PILE INTEGRITY		89	[%]			

PILE DAMAGED:

PILE DAMAGED AT LEVEL - [m] FROM THE LEVEL OF GAUGES - [m]


30-Jan-2008
Soil Data (Asia) Limited
Load (kN)
Displacement (mm)
Pile Top
Bottom
Pile Forces at Ru

TEST RESULT - DYNAMIC PILE TEST (EPC84-6)

CLIENT: PROFICIENCY CONSTRUCTION (MACAU) CO., LTD.

PROJECT: COTAI GALAXY HOTEL & CASINO

DATE: 25/1/2008 *TYPE OF TEST:* ☑ RESTRIKE ☐ END OF DRIVE

EQUIPMENT:

PDA	1423K	ANGLE METER	-	MEASURING TAPE	-
STRAIN GAUGE	EQU. NO.	[F1] 5593	EQU. NO.	[F2] 5594	
	CAL	-	CAL	-	
ACCELEROMETER	EQU. NO.	[A1] 92275	EQU. NO.	[A2] 92276	
	CAL	-	CAL	-	

HAMMER PROPERTIES:

HAMMER TYPE ☑ HYDRAULIC ☐ DROP HAMMER ☐ SPECIFIED

TYPE OF CUSHION ☑ PLYWOOD ☐ RUBBER ☐ SPECIFIED

SERIAL NO. - WEIGHT OF RAM - [ton]

PILE PROPERTIES:

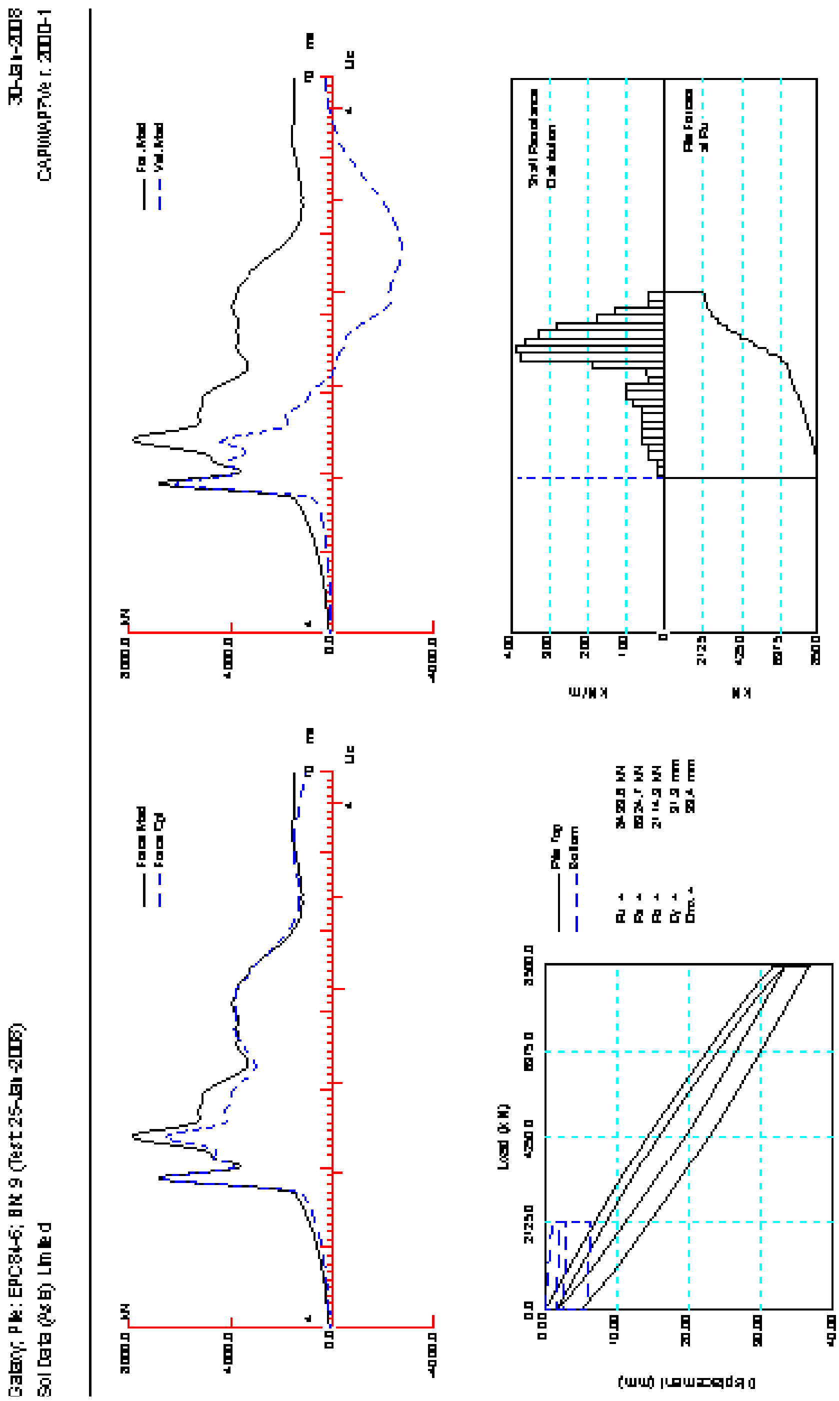
PILE NO.	EPC84-6	L_t	50 [m]	ELASTIC MODULUS	53688 [MPa]
TYPE	CONCRETE PILE	L_e	49.50 [m]	YIELD STRENGTH	80 [MPa]
INCLINATION	[]	L_p	49.00 [m]	REQUIRE CAPACITY	7000 [kN]
AREA	0.192 [m²]	GRADE	80	WAVE SPEED	4500 [m/s]
COATING	NONE	IMPEDANCE	2.29E+03 [kNs/m]	SPECIFIC WEIGHT	26 [kN/m³]

TEST RESULTS:

BLOW NO.		9				
FINAL SET		-	[mm / BLOW]			
NEAREST BOREHOLE RECORD		-				
DATE OF LAST HAMMERING		-				
CASE STATIC BEARING CAPACITY WITH VARIOUS DAMPING FACTORS	J_c = 0.0	10039	[kN]	J_c = 0.5	7916	[kN]
	J_c = 0.1	9614	[kN]	J_c = 0.6	7491	[kN]
	J_c = 0.2	9190	[kN]	J_c = 0.7	7066	[kN]
	J_c = 0.3	8765	[kN]	J_c = 0.8	6642	[kN]
	J_c = 0.4	8340	[kN]	J_c = 0.9	6217	[kN]
MAX. FORCE IN SENSOR LEVEL		8300	[kN]			
MAX. COMPRESSIVE STRESS		43.2	[MPa]			
MAX. TENSILE STRESS		0	[MPa]			
MAX. ENERGY TRANSFERRED		86.6	[kNm]			
HAMMER DROP HEIGHT		-	[m]			
HAMMER RATED ENERGY		288	[kNm]			
ENERGY TRANSFERRED		30%	[%]			
PILE INTEGRITY		88	[%]			

PILE DAMAGED:

PILE DAMAGED AT LEVEL - [m] FROM THE LEVEL OF GAUGES - [m]



TEST RESULT - DYNAMIC PILE TEST (EPC85-1)

CLIENT: PROFICIENCY CONSTRUCTION (MACAU) CO., LTD.

PROJECT: COTAI GALAXY HOTEL & CASINO

DATE: 25/1/2008

TYPE OF TEST: ☑ RESTRIKE ☐ END OF DRIVE

EQUIPMENT:

PDA	1423K	ANGLE METER	-	MEASURING TAPE	-
STRAIN GAUGE EQU. NO.	[F1] 5593	EQU. NO.	[F2] 5594		
CAL	-	CAL	-		
ACCELEROMETER EQU. NO.	[A1] 92275	EQU. NO.	[A2] 92276		
CAL	-	CAL	-		

HAMMER PROPERTIES:

HAMMER TYPE ☑ HYDRAULIC ☐ DROP HAMMER ☐ SPECIFIED

TYPE OF CUSHION ☑ PLYWOOD ☐ RUBBER ☐ SPECIFIED

SERIAL NO. - WEIGHT OF RAM - [ton]

PILE PROPERTIES:

PILE NO.	EPC85-1	L_t	48 [m]	ELASTIC MODULUS	53688 [MPa]
TYPE	CONCRETE PILE	L_e	47.50 [m]	YIELD STRENGTH	80 [MPa]
INCLINATION	[]	L_p	47.00 [m]	REQUIRE CAPACITY	7000 [kN]
AREA	0.192 [m²]	GRADE	80	WAVESPEED	4500 [m/s]
COATING	NONE	IMPEDANCE	2.29E+03 [kNs/m]	SPECIFIC WEIGHT	26 [kN/m³]

TEST RESULTS:

BLOW NO.		5			
FINAL SET		- [mm / BLOW]			
NEAREST BOREHOLE RECORD		-			
DATE OF LAST HAMMERING		-			
CASE STATIC BEARING CAPACITY WITH VARIOUS DAMPING FACTORS	J_c = 0.0	9675 [kN]	J_c = 0.5	7122 [kN]	
	J_c = 0.1	9164 [kN]	J_c = 0.6	6612 [kN]	
	J_c = 0.2	8654 [kN]	J_c = 0.7	6102 [kN]	
	J_c = 0.3	8143 [kN]	J_c = 0.8	5591 [kN]	
	J_c = 0.4	7633 [kN]	J_c = 0.9	5081 [kN]	
MAX. FORCE IN SENSOR LEVEL		9221 [kN]			
MAX. COMPRESSIVE STRESS		48 [MPa]			
MAX. TENSILE STRESS		0 [MPa]			
MAX. ENERGY TRANSFERRED		76.5 [kNm]			
HAMMER DROP HEIGHT		- [m]			
HAMMER RATED ENERGY		288 [kNm]			
ENERGY TRANSFERRED		27% [%]			
PILE INTEGRITY		90 [%]			

PILE DAMAGED:

PILE DAMAGED AT LEVEL - [m] FROM THE LEVEL OF GAUGES - [m]